

004761
Vol. 2

URBAN DOCUMENTATION CENTRE
RESEARCH UNIT FOR URBAN STUDIES
McMASTER UNIVERSITY
HAMILTON, ONTARIO

STEEL ALTERNATIVE GUIDEWAY STUDY

PHASES 1 AND 2

FEASIBILITY REPORT

APPENDICES 8.1 8.2 8.3 8.4

8.5 8.6 8.7



Hamilton-Wentworth Rapid Transit Project

PRE-IMPLEMENTATION PROGRAM


metro canada
limited

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.1

SURVEY OF CURRENT LITERATURE ON

ELEVATED GUIDEWAY CONCEPTS FOR

RAPID TRANSIT SYSTEMS

METRO CANADA LIMITED,
RAPID TRANSIT PROJECT OFFICE,
HAMILTON, ONTARIO

MARCH, 1981

Contents

1) Introduction	Page 1
2) Reports Reviewed	2
3) Literature Survey Synopsis	
Report No. 1 - MTC April 1979	3-4
" " 2 - UMTA July 1979	4-5
" " 3 - ABAM Nov. 1980	5
" " 4 - CSICC Nov. 1975	5-6
" " 5 - AISI	6-7
" " 6 - HSCL	7-9
" " 7 - AISC 1979	10
4) Literature Survey Summary	
Report No. 1 - MTC April 1979	11-21
" " 2 - UMTA July 1979	22-44
" " 3 - ABAM Nov. 1980	45-46
" " 4 - CSICC Nov. 1975	47-60
" " 5 - AISI	61-62
" " 6 - HSCL	63-74
" " 7 - AISC 1979	75-86

1) INTRODUCTION

The Technical Advisory Committee requested Metro Canada Limited to prepare a proposal for the analysis of a steel ICTS guideway in addition to the already contracted analysis of a concrete ICTS guideway. The proposal finally accepted and authorized for implementation included the following general activities and the participants involved:

- (i) Management and Environment - MCL Hamilton
- (ii) Systems Review - MCL Kingston
- (iii) Information and Criteria - ABAM Limited
- (iv) Structural and Aesthetics - Cole Sherman & Associates Inc. - Trevor Garwood Jones
- (v) Noise and Vibration - Wilson Ihrig & Associates Inc.

One of the work statements included in task (i) is a review of previous studies and preparation of a resume, analysis and summary report. The purpose of this review is to determine the scope of recent studies, the general conclusions reached and to provide data that will avoid duplication of work and assist in selecting an appropriate steel cross section for an aerial ICTS guideway.

Due to the time and financial restraints imposed on the study the reports were reviewed by copying what appeared to be the significant and appropriate sections as they related to the present study. These sections were then analyzed, summarized and used as reference documents during the progression of the study as far as they applied to the design of the steel alternative guideway system. The summarized statements and comments will also be used as a check list during preparation of the final report to ensure that all appropriate factors have been addressed and considered.

This report documents the results of the survey of current literature on elevated guideway concepts for rapid transit systems.

2) REPORTS REVIEWED

The following reports have been reviewed and summarized:

- Report No. 1 - An evaluation of Steel and Concrete Guideway Sections for ICTS. Ministry of Transportation and Communications. April 1979. (2.33.2)
- Report No. 2 - AGT Guideway and Station Technology. Guideway and Station Concepts. United States Department of Transportation. UMTA July 1979. (2.33.1)
- Report No. 3 - Trip Report. Visit to modern American transit systems. ABAM Limited. November 26, 1980. (2.51.1)
- Report No. 4 - Proposal for Elevated Steel Guideway Concepts for use in ICTS. Canadian Steel Industries Construction Council and Read-Harris Engineering. November 1975. (2.33.3)
- Report No. 5 - Elevated Structures for Mass Transit Systems. American Iron and Steel Institute. (2.31.6)
- Report No. 6 - Technical Design Proposal for Intermediate Capacity Transit Demonstration System. Volume III. Hawker Siddeley Canada Limited. Prepared by Cole, Sherman and Associates Limited. (2.61.1)
- Report No. 7 - Prefabricated press-formed steel T-Box girder bridge system
Narendra Taly and H.V.S. Gangarao
AISC Journal. Third quarter/1979 (2.51.3)

Note: The Metro Canada Limited, Hamilton office library reference number for the above documents is shown in parenthesis adjacent to each title.

Steel Alternative Guideway Study

Literature Survey Synopsis

The following is a brief synopsis of the literature summaries contained in this report. Where figure or table numbers are indicated, reference should be made to the copies contained in the literature summary section of this report.

Report No. 1 MTC April 1979

A formative study of three potential guideway sections is presented for application to the ICTS system developed by the UTDC. A large number of candidate sections for single guideway was examined. Many of the sections were summarily dismissed usually for lack of structural capability, inflexibility of application, or complexity of construction. The summary sections were basically in three groups: channels, box girders and girder slabs. The channel section was considered inappropriate for ICTS and this report considers only box girders and girder slabs.

Candidate box girder sections are shown in figure 3.1 and candidate girder slab sections are shown in figure 3.2. The key elements in the report that should be considered during the evaluation of the steel alternative guideway study are listed as follows:

- Sections B4, B5 and G5 have less difficulty in providing spans of 140 feet in a single unit with a depth of about 6 to 7 feet.
- The trapezoidal boxes B2, B3 and B5 at long spans are considered most appropriate for high quality urban integration.
- The Urban Design Study suggests different structural approaches in different situations and demonstrates clearly the likelihood of special structures for dealing with many of the tight problems of urban fit.
- It is possible that an ICTS route might consist of a number of relatively short segments of different types of construction.
- The beam sections B1, B2, B4 and B5 and the girder sections G2, G4, and G5 may be made curved.
- The performance of composite steel box girders as torsionally stiff aerial guideway elements is comparable to that of concrete box girders. Generally, they are competitive in price. Their main disadvantages, however, are the propagation of noise, and maintenance. Even when maintenance free atmospheric resistant steel is used, the consequences of rusty run-off water could become both unsightly and costly to control. However, their construction in particular areas could be cost-effective, especially when long spans are planned in guideway portions where noise and drippings are of little concern.
- The steel box girder sections can be applied in spans between 120 and 170 feet. For simple spans the economic span range is between 120 and 140 feet, while steel box girders are common in highway construction, they might be more limited in application to transit systems mainly due to requirements for shorter spans within urban areas.

-Any cost estimate for a proposed elevated guideway based on generalized unit costs, not considering site-specific and contractual details, is likely to be unrealistic and low. Guideway costs are more significantly reduced by careful planning, attention to detail and standardization in design, tender and contract preparation, competition in bidding, and other such facts, rather than by cross-section and span length relationships alone. On any given route, cross-sections of different types may be appropriate for different route segments.

Report No. 2 UMTA July 1979

Guideway and station concepts are presented for automatic guideway transit (AGT) systems with emphasis on lowering the cost and reducing the implementation time associated with guideways and stations. The guideway concepts work includes a discussion for various materials and construction techniques and assesses their applicability to AGT.

The following are the key elements of the report for consideration:

- Design activities and construction techniques that tend to reduce implementation time are:
 - 1) Off-site fabrication
 - 2) Simplicity of design
 - 3) Longer spans
 - 4) Optimum integration of subsystems
 - 5) Standardization
 - 6) Modulization
- Two optimum subsystem integration techniques have been identified. The first technique uses design refinements to minimize the amount of assembly time. The second increases the amount of off-site subsystem integration.
- The use of standard specifications and standard techniques reduces both implementation time and cost.
- Welding and bonding under field conditions must be accomplished with a high degree of reliability under a variety of climatological and environmental conditions.
- Materials selected must not be adversely affected by wind, rain, snow, ice, temperature, lightning strikes, or sunlight. They also must not be sensitive to contaminants such as leaking oil, chemicals used on the guideway, and stray electrical currents.
- It is desirable that transit guideways not require frequent inspections: any material tending to require frequent inspection and maintenance should be avoided.
- High strength steel is extremely cost effective if it can be used properly.

- The guideways associated with 30 systems were reviewed. Six existing AGT systems were selected for analysis. The primary objective was reduction of guideway costs and construction time.
- Figure 17 shows the beam shapes for the six selected existing guideways that were assessed.
- The Sections shown in Figure 47 are also used in other types of construction, the required tooling is commonly available thereby saving both time and money.
- The specialty shapes shown in Figure 48 are divided into two groups: box sections and channel sections. So they can provide an integrated ride surface, these shapes offer the possibility for a more efficient design. The channel and box sections are capable of providing an aesthetically pleasing appearance, have long-span capability, good torsional properties for curved beams and the combination of these characteristics make it an efficient cost effective choice.

Report No. 3 ABAM November 26 1980

A visit was made to five modern American systems to improve the background for the ICTS development program particularly related to trackwork.

The following recommendations are based on the results of the trip as they apply to ICTS guideway:

- Use continuous welded rail
- Rail expansion joints minimal
- Consider Hixson fastener
- Minimize loss of bolt preload
- Construction Quality Control
- Second pour installation method
- Standardize ICTS special trackwork geometrics
- Alternatives of switch/frog machines to minimize size of structure
- Hixson fastener type for special trackwork

Report No. 4 CSICC and Read-Harris Engineering November 1975

This report covers the preliminary engineering which has been carried out to evaluate several concepts including several variations of steel box girders. These are rectangular, trapezoidal and semi-octagonal shapes.

A truss type supporting structure, fabricated using hollow structural sections is presented with the first method having a cast in place deck and the second using precast concrete deck segments. The second method would have several advantages.

- No forming would be required
- Quality control would be much improved
- Savings in steel support structure weight.

The following general statements are taken from the report:

- In either the cast in place or the precast system, allowances can be made for curvature in the guideway.
- The cast in place system allows for more precise construction of curves.
- Strike a balance between the acceptable and desirable aesthetic acceptability vs unit cost of construction of the deck.
- For the fabrication of straight line girders, the fabrication and utilization of materials can be very efficient.
- No 1 - Type A girder is the most efficient in terms of fabrication. Type B girder is a trapezoidal box section with reduced bottom flange plate width which allows for more efficient use of flange plate material.
- The use of tubular or hollow structural sections has become more evident (Plate 4).
- The girder type (Plate 5) should be fabricated at prices comparable to box girders.
- Fabricating curved girders would result in significant waste material.
- Concrete piers are generally used for bridges and could be acceptable material for steel guideway system. All piers could be precast. Another study should be carried out on the use of steel piers.
- Guideway System Corrosion Protection:
 - protective coating would have a life expectancy of 20 years.
 - Atmospheric corrosion resistant steel would require concrete surfaces to be protected to prevent staining. This problem would be minimized if steel piers were used.
 - Conclusion is that the most pleasing structures would be concept No. 3 and Concept No. 1A in that order.

Report No. 5 American Iron and Steel Institute

This book is aimed primarily at the structural engineer faced with the task of designing an elevated guideway. The emphasis is on aspects which distinguish transit guideways from highway and railway bridges. Design methods are outlined with particular emphasis on the frequency components of vibration and random vibrations.

Types of construction are described and the following comments and statements relate to the steel guideway alternative study:

-An alternative to twin I - girders is a box girder. Box girders are frequently desired for their pleasing appearance. The webs may be vertical or inclined.

-Guideways consisting of simple spans are not recommended where there is any difficulty in achieving ride quality.

-Continuous span units have substantial advantages over simple spans relative to vehicle vibrations.

-Slabs can be cast on the simple spans either in place or prior to erection provided suitable splice details are developed.

-Selection of span length is affected by many considerations:

- crossing of streets
- physical obstacles
- general appearance of structure
- transportation and erection

-When other constraints leave room for choice, economy is the governing factor.

-Several methods exist for fabrication of a curved girder.

Report No. 6 Hawker Siddeley Canada Limited

This report was prepared to assist in the selection of a guideway for the ICTS demonstration project proposed for the Canadian National Exhibition grounds in Toronto. The feature of the system which had a major impact on guideway design was the single-sided linear induction motor and the development of a magnetic levitation vehicle concept. The approach taken to the guideway design was first to consider the various alternative beam sections, materials and construction methods available for guideway application. Applying this approach the choice of guideway sections was narrowed to basic beams for which span/cost ratio curves were plotted, leading to optimum alternatives for varied applications.

Factors considered in the comparative evaluation include:

- Economy
- Aesthetics
- Geometry of the alignment
- Problems at switches
- Restraints on span lengths
- Construction Industry familiarity with construction of various segments.
- Anticipated competitive condition of the construction market.
- Availability of local construction materials
- Speed and ease of construction
- Schedule restraints
- Maintenance costs

- Noise generation
- Freedom from interference with automatic vehicle control system
- System expansion
- Ability to meet required dimensional tolerances
- Ease of snow removal
- Construction of curvature and superelevation

The following comments and statements were made in the report in the sections dealing with the various alternatives:

Structural Steel - box section

- This is the simplest solution in terms of meeting the geometric requirements and varying span lengths, by shop fabrication from steel plates.
- It has merit in speed and ease of construction.
- Use of atmospheric corrosion type steel leads to favourable maintenance costs, except for staining of sub-structure which must be corrected or prevented.
- A serious disadvantage is the probability of vehicle activated vibrations generating and propagating resonance in the girders.
- Countering vibrations generated may escalate the cost.
- Control system interference may be a problem.
- Custom fabrication for curved and superelevated members is costly.
- Field connection or adjustment of any misalignment is difficult.
- The steel box section is considered to have poor visual appeal, particularly in the early years of weathering.

Open box girder, sloped sides, cast in place concrete deck.

- Of the structural steel concepts this has the greatest aesthetic appeal.
- It is structurally efficient due to the narrow bottom flange.
- Cost escalates dramatically when combined curved and superelevated members have to be fabricated.
- This section is suited to longer span lengths on straight stretches of guideway (in the order of 120 feet).

Open box girder, vertical sides, cast in place concrete deck.

- The advantages and disadvantages applicable to the above alternatives also apply to this solution, but with the following modifications.
- It has less aesthetic appeal.
- Construction costs will be less in curved alignments.

For aesthetic and cost reasons, but primarily due to anticipated vehicle control system interference problems, the structural steel alternatives are not suitable for the present applications.

Precast Concrete

- Two major precast manufacturers conceded that all switch sections would require to be cast-in-place concrete.
- Few applications for standardization suited to factory mass production exist on the system.
- When mixing concrete casting methods it is feared that matching of colour and surface would be difficult.
- Development and fabrication of such framework is costly.
- Curvature of the elements would restrict the lengths that can be shipped.
- This could be a viable solution if long tangent sections were the rule rather than the exception.
- Because of the many disadvantages of the precast concrete method when applied to the demonstration system this method is eliminated from present consideration.

Cast in place concrete

- This is the selected construction method.
- Merits are its suitability for building to any curvature, superelevation and varying width such as at switches, by adjusting the framework under precise engineering control on site, lead time is minimal and the contractor can quickly become mobilized.
- Competitiveness of the general construction industry is keen for this kind of work.
- Will require proper engineering of falsework to accommodate existing traffic routes.
- Advantageous for the casting-in of anchor bolts, electrical conduits and inserts.
- A prime consideration in determining the shape of the guideway is that of aesthetics.
- Concrete surfaces should be homogeneous.
- Steel, high density plywood or fibreglass-coated forms should be used.

Piers

- Reinforced concrete circular section is the most effective and economical.

Deck Topping and Finish

- Concrete topping with a hardened finish of at least 3" thickness will be applied over the concrete surface to the precise finished elevations.

Report No. 7 AISC 1979

An all-steel, all-welded box-girder for HS20-44 loading has been presented. For spans up to 65 ft., the proposed superstructure is light and weighs only about 40-45 psf, as compared with over 100 psf for a conventional slab-stringer bridge system. The system is developed from the viewpoints of segmental assemblage, low initial and maintenance costs, and ease of future expansion.

As an alternative to the steel plate deck, a design with precast, prestressed concrete deck that acts compositely with the steel section is also presented. This system is developed from the viewpoints of segmental assemblage, low initial and maintenance costs, and ease of future expansion.

As an alternative to the steel plate deck, a design with precast, prestressed concrete deck that acts compositely with the steel section is also presented. This system weighs about 85 psf. It is lighter than the conventional slab-stringer system, and lends itself better to modular construction.

Six different girder sections have been presented with a maximum span capacity of 65 ft. The girders have widths of 6 and 8 ft. and depths of 30, 36 and 42 in. A suitable combination of these sections can be used to achieve the required bridge width. By increasing the depth of the girders, the system can be used for longer spans.

The proposed system has been designed under the constraints of 6 and 8-ft. wide girder units, from the viewpoint of modular systems. However, further optimization can be achieved by increasing the flange widths to 10 or 12 ft. This would decrease the unit superstructure weight to about 35 psf. With deeper girders and thicker webs and flanges, the system is adaptable to longer spans.

Steel Alternative Guideway Study

Literature Survey Summary

The following data have been copied directly from each of the reports reviewed. Only those portions of the reports that seemed appropriate to the steel alternative guideway were copied for analyses and integration with the other phases of the study being undertaken concurrently.

Report No. 1 MTC April 1979

An evaluation of steel and concrete guideway sections for ICTS.

A parametric study of three potential single-track guideway sections is presented for application to the ICTS system developed by the UTDC. These sections comprise a double-Tee beam using modified Highway I-girders, a prestressed concrete trapezoidal box girder originally developed for highway bridges and a concrete-steel composite box girder predominantly used for long span highway bridges. The performance of a purely double-Tee single track guideway as developed and tested for the Miami Transit System is extrapolated from available data and published papers.

A large number of candidate sections for single guideway was examined. Many of the sections were summarily dismissed, usually for lack of structural capability, inflexibility of application, or complexity of construction. The surviving sections were basically in three groups: channels, box girders and girder slabs. The barrier wall of a channel section, being a primary structural element, is susceptible to damage from a derailed vehicle which might drastically reduce its structural adequacy, and the range of span for which a constant barrier wall height and width is both feasible and efficient is quite limited. For these reasons, the channel section is considered inappropriate for ICTS and this report considers only box girders and girder slabs.

The potential sections of this study are considered in both steel and concrete, with particular attention paid to the former since it has proven cost-competitive in a number of recent bridge and guideway applications and it provides alternative urban

integration opportunities to concrete.

Candidate box girder sections are shown in Figure 3.1 and candidate girder-slab sections are shown in Figure 3.2. The sections B4, B5 and G5 have less difficulty in providing the 140 foot (42.7m) span required by UTDC in a single unit with a depth of about 6 to 7 feet. UTDC's preferred sections for this study are the trapezoidal boxes B2, B3 and B5. The trapezoidal box section at long spans is considered most appropriate for high quality urban integration.

While the original ICTS thinking perhaps was to minimize cost by standardization of elements in a single large guideway production contract, the Urban Design Study suggests different structural approaches in different situations and demonstrates clearly the likelihood of special structures for dealing with many of the tight problems or urban fit. It is possible that an ICTS route might consist of a number of relatively short segments of different types of construction. Opportunity with this approach for many small construction contracts.

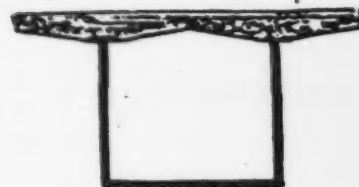
The beam sections B1, B2, B4 and B5 and the girder sections G2, G4 and G5 may be made curved.

Lowest cost to be in the span range of 35-50 feet (10.7-15.2m). The cost variation over quite a wide span range is fairly low and that essentially the same span range is found optimal for greatly different vehicle weights, structural designs and foundations.

The performance of composite steel box girders as torsionally stiff aerial guideway elements is comparable to that of concrete box girders. Generally, they are competitive in price. Their main disadvantages, however, are the propagation of noise, and maintenance. Even when "maintenance free" atmospheric resistant steel is used, the consequences of rusty run-off water could become both unsightly and costly. However, their construction in particular areas could be cost-effective, especially when long spans



(B1)



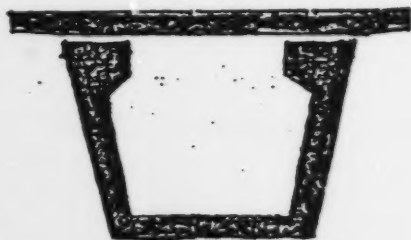
(B4)

VERTICAL WEBS

(B2)



(B5)

INCLINED WEBS

(B3)

TrapezoidalPRESTRESSED/POST-TENSIONED
CONCRETECOMPOSITE STRUCTURAL
STEEL-CONCRETE

Fig. 3.1 Guideway Cross-Sections Box Girders



(G1) Precast



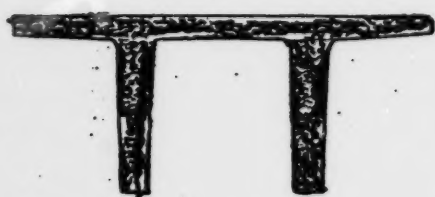
(G4) Rolled

STANDARD SECTIONS

(G2) Modified Standard



(G5) Built-up

MODIFIED SECTIONS

(G3) Double-Tee

PRESTRESSED/POST-TENSIONED
CONCRETECOMPOSITE STRUCTURAL
STEEL-CONCRETE

Fig. 3.29. Guideway Cross-sections Girder-Slab

are planned in guideway portions where noise and drippings are of little concern.

While boxes with vertical webs are easier to fabricate inclined webs could be more aesthetically suitable. However, as far as the design and detailing aspects are concerned, there is very little difference between the two as long as the inclination of the webs does not exceed about 1 in 4.

A 5 foot (1.5m) deep girder permits a simple span of 125 feet (38m) with a corresponding live load deflection of $1\frac{1}{4}$ inch (3.2 cm). The same depth can span up to 145 feet (44m) with a more relaxed deflection limitation of $1\frac{7}{8}$ inch (4.8cm).

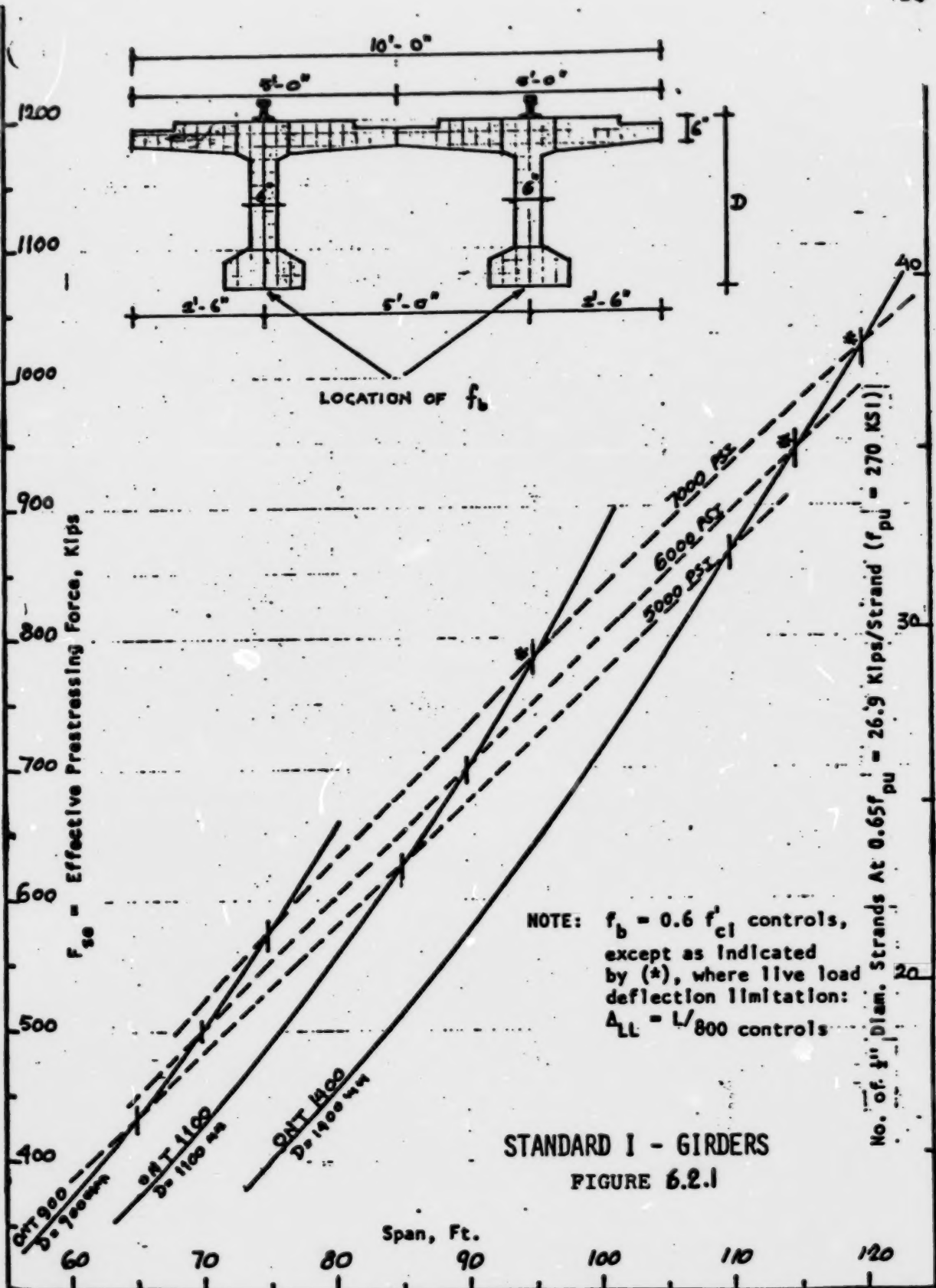
See Figures 6.2.1, 6.5.1, 6.6.3 and 6.6.4. The steel box girder sections can be applied between 120 feet (36m) and 170 feet (51m).

The trapezoidal section used for highway bridges as shown in Figures 6.6.1, 6.6.3 and 6.6.4 looks very promising for ICTS applications.

For a span of 100 feet (30m) two concrete I-girders cost \$130/LF (\$425/m) a trapezoidal box costs \$100/LF (\$328/m) and a steel box costs \$180/LF (\$590/m) in 1978 Canadian dollars as derived from MTC bridge cost records.

Steel box girders - for simple spans, the economic span range is between 120 feet and 140 feet (36m and 42m) with a minimum point at 135 feet (41m), costing \$46/SF (\$495/m²). The rate of change in cost for the continuous construction is more pronounced, ranging between \$41/SF (\$441/m²) at 120 feet (36m) and 160 feet (48m) down to a minimum of \$36/SF (\$388/m²) at 140 feet (42m). While steel box girders are common in highway construction, they might be more limited in application to transit systems mainly due to requirements for shorter spans within urban areas.

The finished cost of a guideway depends somewhat upon the superstructure cross-section and detailing of its fabrication and



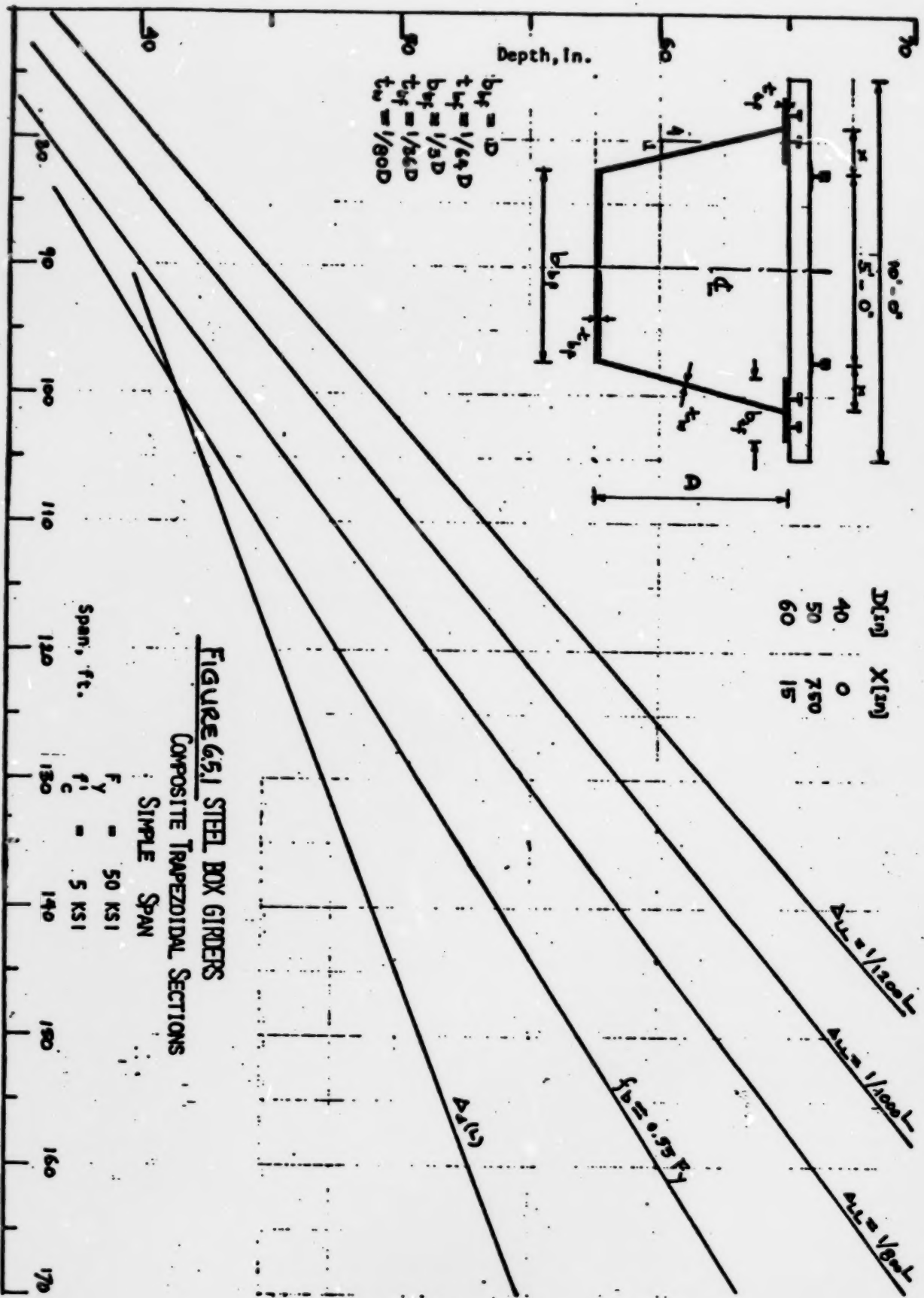
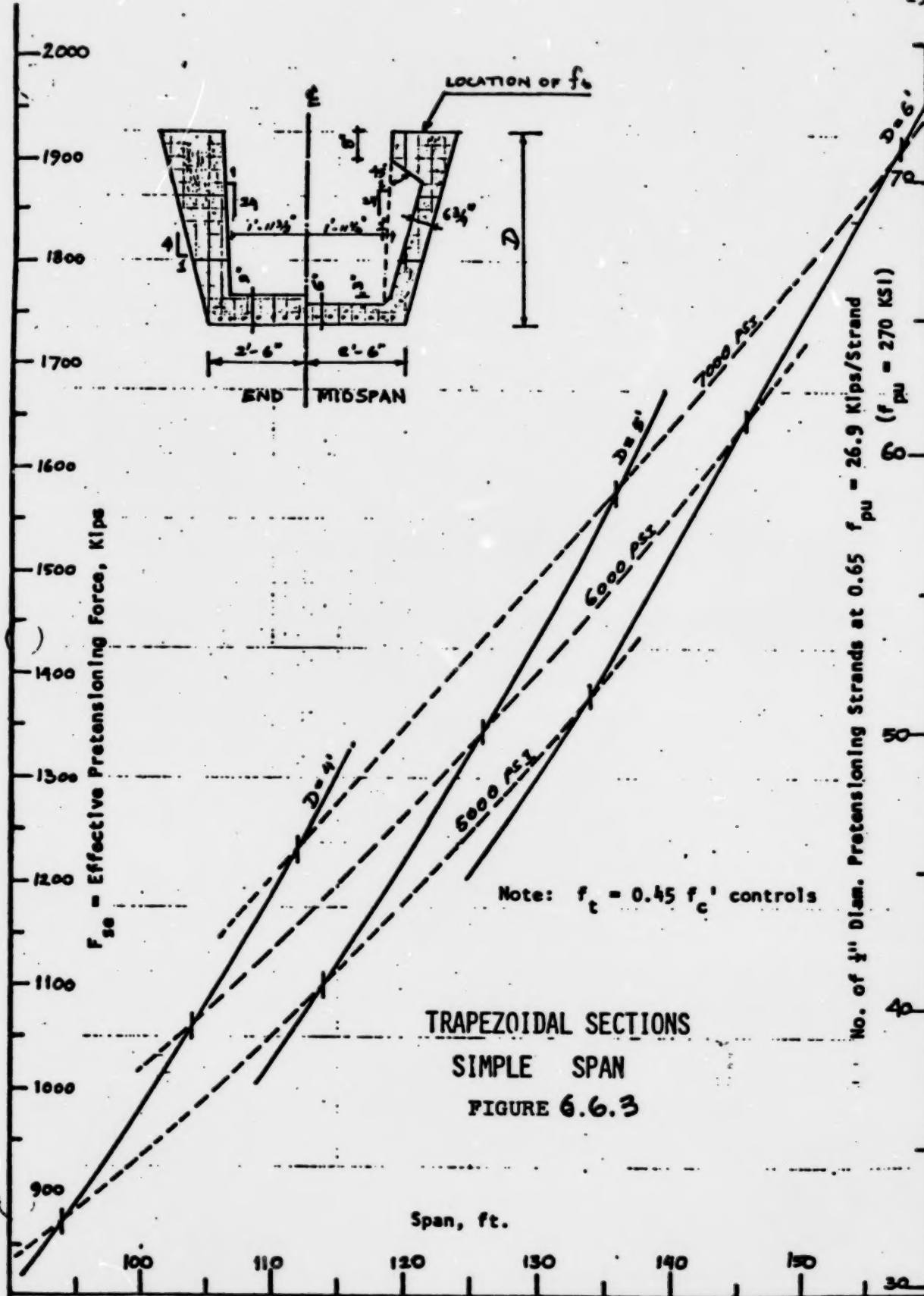


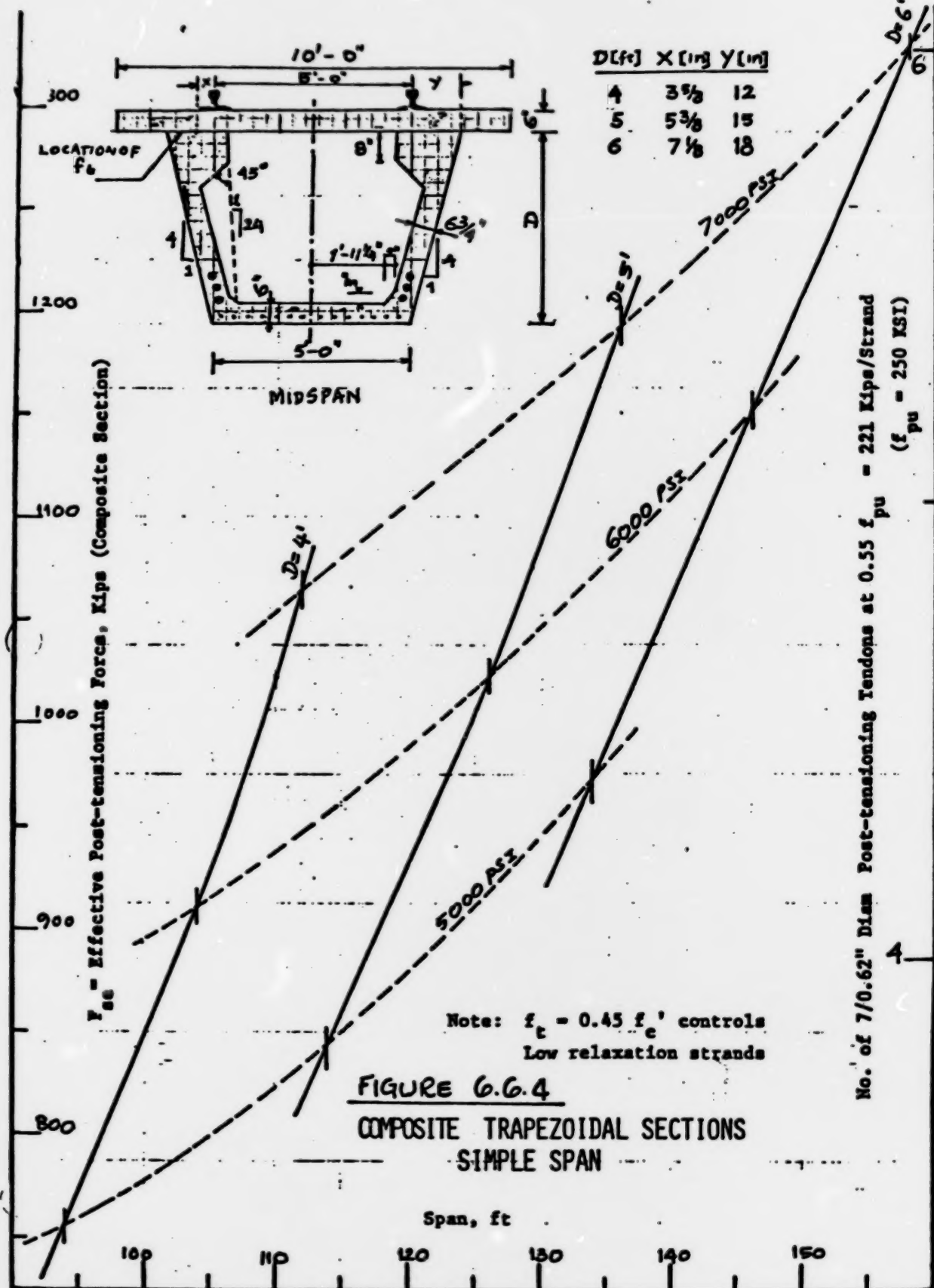
FIGURE 6.5.1 STEEL BOX GIRDERS

COMPOSITE TRAPEZOIDAL SECTIONS

SIMPLE SPAN

$F_y = 50 \text{ KSI}$
 $F_c = 5 \text{ KSI}$





assembly. It depends strongly upon the substructure, sub-surface and other interferences with construction and contractual terms and conditions. Any cost estimate for a proposed elevated guideway based on generalized unit costs, not considering site-specific and contractual details, is likely to be unrealistic and low. Guideway costs are more significantly reduced by careful planning, attention to detail and standardization in design, tender and contract preparation, competition in bidding, and other such facts, rather than by cross-section and span length relationships alone.

On any given route, cross-sections of different types may be appropriate for different route segments.

Report No. 2

AGT Guideway and station technology. Guideway and station concepts US. DOT UMTA July 1979

Guideway and station concepts are presented for automatic guideway transit (AGT) systems with emphasis on lowering the cost and reducing the implementation time associated with guideways and stations. The guideway concepts work includes a discussion of various materials and construction techniques and assesses their applicability to AGT.

Trend Band analyses - unit costs increase with an increase in beam width, - unit cost increases as dead load increases, - total unit guideway cost increases with an increase in vehicle live load, - the unit cost decreases as the span length increases for the single-lane guideways.

Design activities and construction techniques that tend to reduce implementation time are 1) off-site fabrication, 2) simplicity of design, 3) longer spans, 4) optimum integration of subsystems, 5) standardization, and 6) modularization.

Two optimum subsystem integration techniques have been identified. The first technique uses design refinements to minimize the amount of assembly time. The second increases the amount of off-site subsystem integration. The use of standard specifications and standard techniques also reduces both implementation time and cost. Workability - material must be workable, that is, possess the ability to be shaped, bonded, or welded. Welding and bonding under field conditions must be accomplished with a high degree of reliability under a variety of climatological and environmental conditions.

Materials selected must not be adversely affected by wind, rain, snow, ice, temperature, lightning strikes, or sunlight. They also must not be sensitive to contaminants such as leaking oil, chemicals used on the guideway, and stray electrical currents.

It is desirable that transit guideways not require frequent inspection; any material tending to require frequent inspection and maintenance should be avoided.

Steel is used in plate and rolled shapes as the primary element for structures and also as high strength cables in tension applications.

Concrete is an alternative to steel as a baseline material. One aspect of concrete worthy of future investigation, is its high strength capacity.

High strength steel is extremely cost effective if it can be used properly.

Concrete can be expected to be marginally more cost effective than A-36 steel.

The guideways associated with 30 systems were reviewed. The assessment and updating of selected existing guideways provide a baseline for the new guideway and innovative guideway concept development activities. Six existing AGT systems were selected for analysis. Only the elevated portions of each of these guideways are analyzed. The primary objective of this analysis is to find ways of reducing guideway costs and construction time by estimating the effects changes in design details or construction techniques would have had on the six existing guideways. Figure 17 shows the selected existing guideways assessment matrix.

Figure 47. These sections are also used in other types of construction, the required tooling is commonly available and by eliminating the need to manufacture new tooling and forms, both time and money are saved. The use of standard shapes also reduces orientation and start-up times because of current industry familiarity. This familiarity also lowers costs through more competitive bidding and lower contingencies.

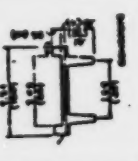
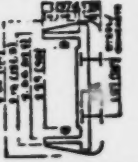
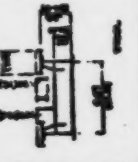
STATION PARAMETERS	RAILROAD CROSSING ALIGNED	RAILROAD YAW CROSSING	METROPOLITAN STATION AND	INTERURBAN STATION	INTER- CITY CROSSING	PRESTRESS STRUCTURAL CROSSING
RAIL ROAD						
STRUCTURE	Concrete	Concrete	Concrete	Steel and Concrete	Steel and Concrete	Steel
CONSTRUCTION TECHNIQUES	Plant Precast a Prestressed	Plant Precast a Prestressed Field Post- Tensioned	Plant Precast a Prestressed a Prestressed	Field Construction Composite Action	Field Construction Composite Action	Prestressing in office, Field Welded Construction
STRUCTURE CERTAINLY	Slab-Open Continuous, Single Lane	Slab-Open Continuous, Single Lane	Slab-Open Continuous, Single Lane	Slab-Open, Slab-Post, Slab	Slab-Open Continuous, Single Lane	Polly Continuous, Single Lane
BEAM POSITIVE MOMENT CONNECTION	Reinf. Steel in Cast-In-Place Joints	Reinf. Steel, Reinforced in Post-Tensioning Joints	Welded Continuous	Not Required	Continuous Reinforced or Reinforced or Reinforced	Continuous Reinforced or Reinforced or Reinforced
BEAM NEGATIVE MOMENT CONNECTION	Continuous Reinf. Steel in Sillings	Continuous Post-Tensioned Reinforcement	Continuous Reinf. Steel in Sillings	Not Required	Continuous Reinforced or Reinforced or Reinforced	Continuous Reinforced or Reinforced or Reinforced

Figure 17. Selected Existing Guideways Assessment Matrix

SYSTEM PARAMETER	RAILROAD-ROAD INTERSECTION ALIGNMENT	RAILROAD TYPICAL CURVES	INTERSECTION TYPICAL AND	INTERSECTION TABLE 1	ARCH COLUMNS	REMARKS AND
ROAD TO CROSS RAILROAD	EXP. JTS. - EVID. & PLANS FOR EXP. JTS. - MAIN INTERSECTION	EXP. & PLANS FOR EXP. & JTS.	EXP. & PLANS FOR EXP. & JTS.	?	?	MAIN RAILROAD TO CROSS VIA SPICE PLANS
TYPICAL SPAN LENGTH (ft)	21.6	16.3	16.3	20.0	22.0-25.0 25.0	20.0
ROAD TO CROSS RAILROAD	16.0	20.0	16.0	11.0	22.0	20.0
REMARKS ON RAILROAD SECTION (ft)	46 ft. for 21.6 span	46 ft. for 16.3 span	41 ft. for 16.3 span	5 ft. for 7.0 span	46 ft.	?
ARCHITECTURAL DETAILS	Self Lined, Light Reinforced Flat	Architecturally Shaped Beam, No Special Finish	No Special Finish	Weathering Steel Flat, Reinforced	Weathering Steel Beam, Reinforced	Weathering Steel Beam and Columns
VEHICLE CROSS SECTION (ft)	91.6	64.2	104.6	12.6	100.7	94.0
VEHICLE CROSS SECTION	Top Surface of Road Beam	.68 ft. Cantilever Plan Concrete Slabbing	.40 ft. Cantilever Plan Concrete Slabbing	.22 ft. Cantilever Plan Concrete Slabbing	.13 ft. Cantilever Plan Concrete Slabbing	Architectural Steel Beam, Plan Concrete Top Surface

Figure 17. Selected Existing Guideways Assessment Matrix (continued)

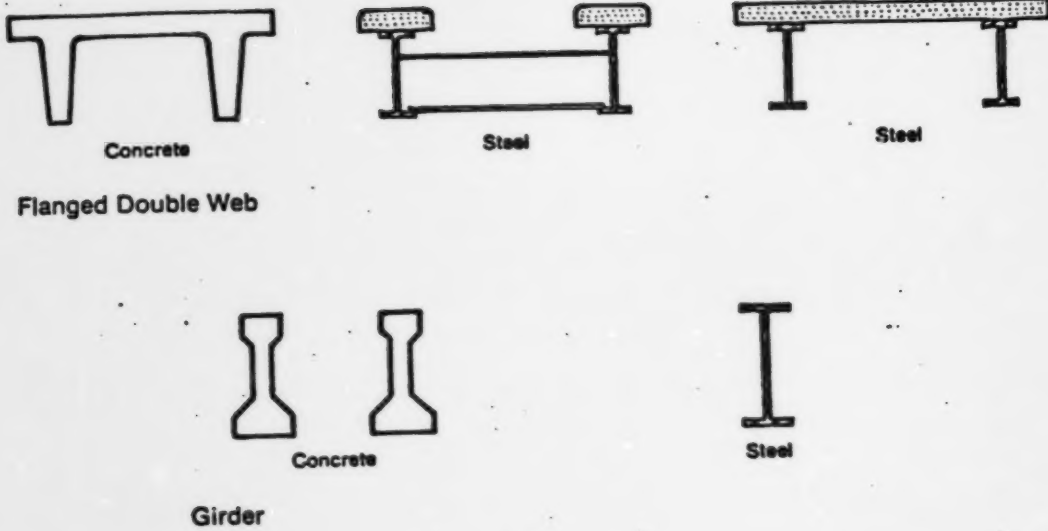


Figure 47. Standard Construction Industry Shapes

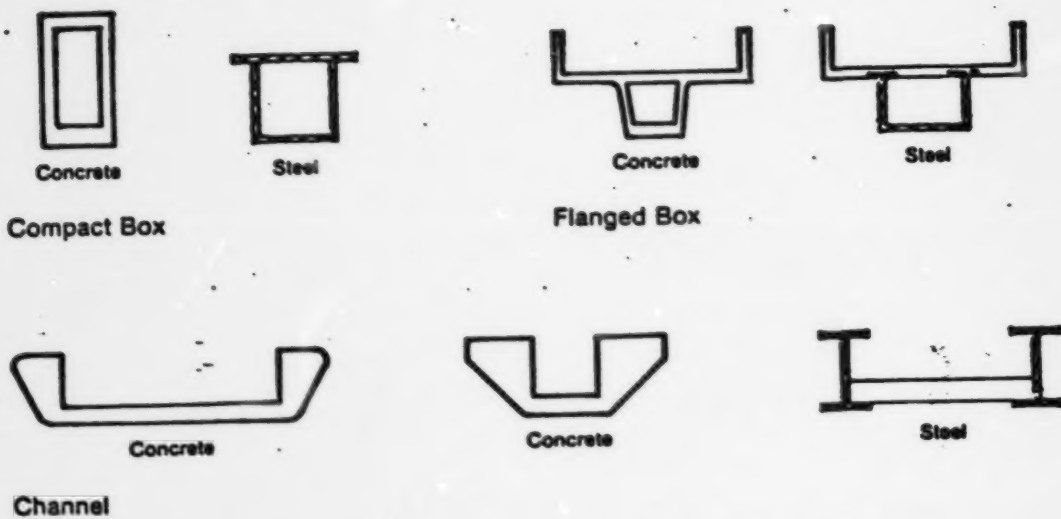


Figure 48. Specialty Shapes

Figure 48. Specialty shapes are divided into two groups: box sections and channel sections. As they can provide an integrated ride surface, these shapes offer the possibility of a more efficient design. In addition, both the channel and box sections are capable of providing an aesthetically pleasing appearance, which is often important in the selection of a guideway. One of the main advantages of the box section is its long-span capability. It also has good torsional properties for curved beams and the combination of these characteristics make it an efficient and cost-effective choice.

The preferred steel box configuration is shown in Shape A of Figure 57. Six optional configurations for the composite flanged box are shown in Figure 59. The steel box is prefabricated and the composite concrete deck is cast-in-place. The preferred configuration is Shape A.

The primary purpose of concept development is to provide design direction for least-cost guideways.

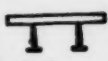
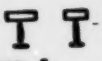
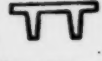
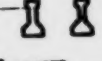
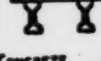
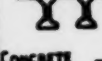
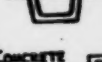
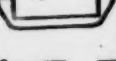
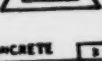
Basic steps of the methodology:

- 1) Check system requirements
- 2) Prepare a preliminary beam design
- 3) Estimate the beam cost
- 4) Estimate the scenario cost

The first step is a check of the basic system requirements, which are divided into three categories: 1) functional requirements, 2) physical requirements, and 3) site integration requirements.

The second step in the guideway evaluation methodology is preliminary beam design (Figure 62). The preliminary design is completed to the detail necessary to generate material quantities for cost estimating.

An estimation of the in-place beam cost is the third step in the guideway concept development and evaluation. A comparison of the estimated and actual as-built costs for each guideway, escalated to 1978 dollars, is shown in Table 20.

FLANGED DOUBLE WEB	 STEEL & CONCRETE ☒	 STEEL & CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒
SINGLE GIRDER	 CONCRETE ☒	 STEEL ☒	 CONCRETE ☒	 CONCRETE ☒			
COMPACT BOX	 STEEL ☒	 STEEL ☒	 STEEL ☒	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒
FLANGED BOX	 STEEL ☒	 STEEL & CONCRETE ☒	 STEEL & CONCRETE ☒	 STEEL & CONCRETE ☒	 CONCRETE ☒		
CHANNEL	 CONCRETE ☒	 CONCRETE ☒	 CONCRETE ☒	 STEEL & CONCRETE ☒	 CONCRETE ☒		

Key: ☒ Indicates Eliminated from Further Concept Development
☒ Indicates Included for Further Concept Development

Figure 49. Beam Shape Matrix

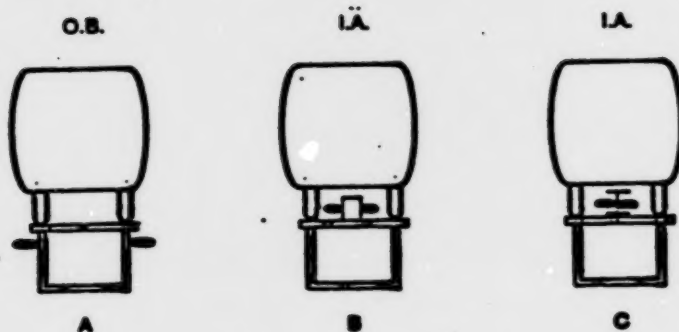


Figure 57. Steel Box

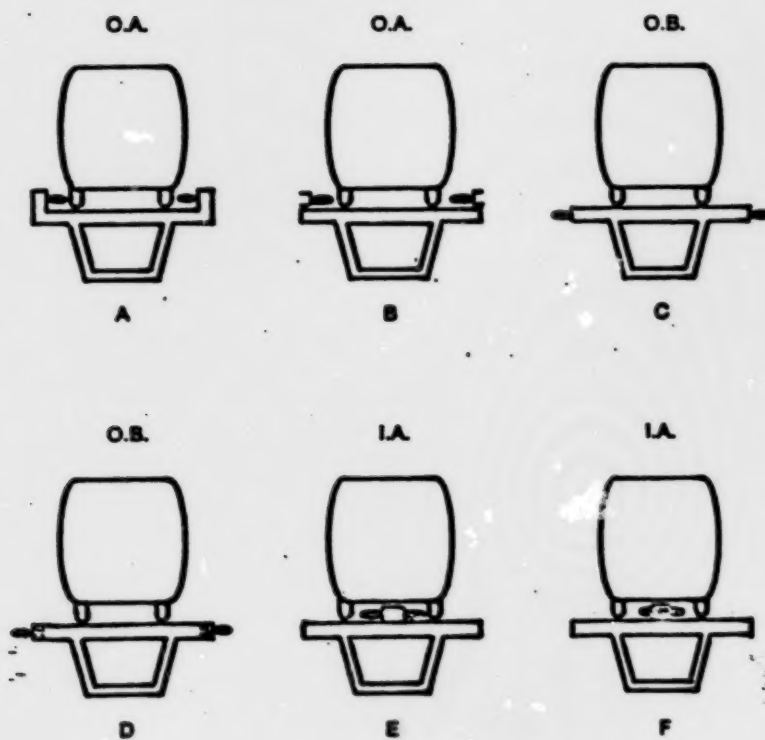


Figure 58. Concrete Flanged Box

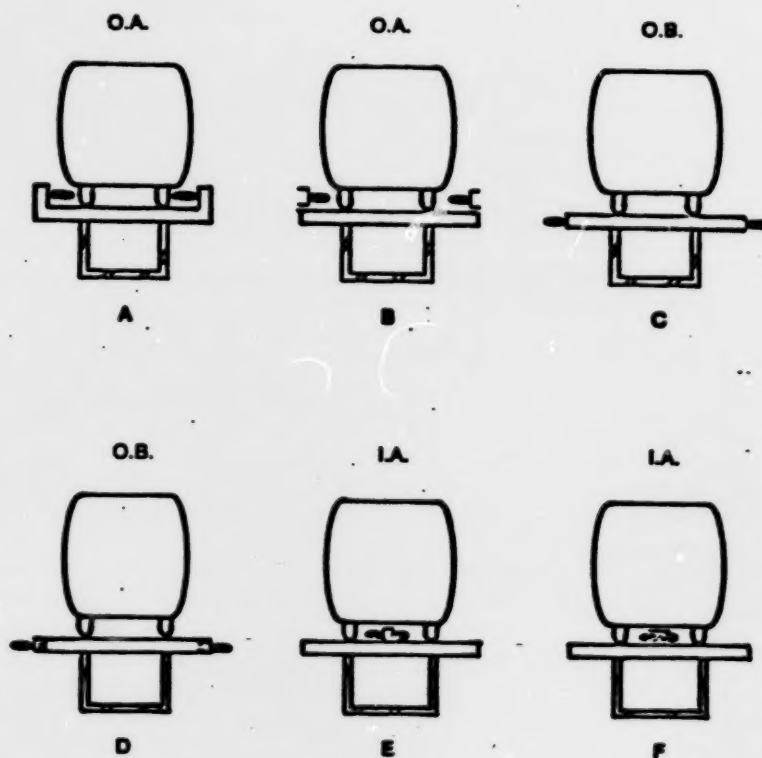


Figure 59. Composite Flanged Box

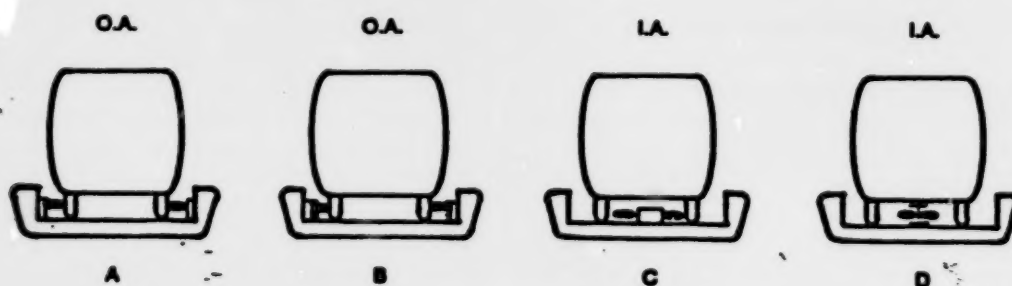


Figure 60. Concrete Channel

STRUCTURAL. **System Loads:**

- Structure Dead Load (D.L.)
- Vehicle Live Load (L.L.) - 5,800 N/m (small vehicle)
 - 11,500 N/m (medium vehicle)
 - 17,500 N/m (large vehicle).
- Impact - assume 10 percent of L.L.
- Centrifugal Force - assume 10 percent of L.L.

. **Environmental Loads:**

- Wind - 960 Pa

. **Serviceability:**

- Maximum Concrete Stresses - tension $250 \sqrt{f_c}$
 - compression $.45 f_c$
- Maximum Steel Stresses - AASHTO "Standard Specifications for Highway Bridges"

MATERIALS

- . Concrete - $f_c = 34.47 \text{ MPa @ 28 days}$
- . Steel - ASTM A36

FRAMING CONCEPT

- . Six-Span Continuous Bridge

Figure 62. Guideway Concept Design Criteria

Table 20. Cost Estimating Procedure Verification

Item	Toronto Zoo	Dallas-Fort Worth Airport	Disney World Monorail	Fairlane
Length of Elevated Guideway, lane-m	2,020	4,170	10,240	988
Estimated Unit Costs (1978 \$)				
Elevated Guideway, \$/lane-m	1,090	1,260	1,470	1,320
Guideway Without Footings, \$/lane-m	830	950	780	1,120
Curved Beam, \$/unit	10,355	19,970	17,450	18,385
Straight Beam, \$/unit	10,670	19,060	15,960	16,670
Single Lane Column, \$/unit	3,345	3,160	5,620	2,920
Dual Lane Column, \$/unit	-	-	10,425	-
Spread Footing, \$/unit	4,390	2,500	-	8,130
Caisson Footing, \$/unit	6,030	4,670	-	3,620
Pile Footing, \$/unit	-	-	26,830	-
Total Estimated Cost, \$000	2,202	5,254	15,081	1,307
Actual Cost Escalated, \$000	2,274(a)	4,685(b)	14,853(c)	1,279

(a) Contractor's as-built construction costs.

(b) Contractor's fixed price contract with system supplier.

(c) Designer's records.

(d) Based on Bradley Field precast subcontract value plus data from (6028) and applied to Fairlane based on the ratio of the elevated guideway lengths of 930 m for Bradley and 988 m for Fairlane.

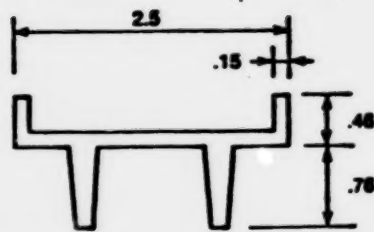
Table 21. Guideway Cost Estimate Comparison

Vehicle Supplier	Guideway Shape	Cost Estimates, 1978\$	
		Contractors	Engineer
#1	Channel Shaped (Complete Guideway)	1,775,700 to 2,346,500	1,879,200
#2	Concrete Box Girders (Girders Only)	661,354	650,400

The fourth step in guideway concept development and evaluation is the generation of a guideway system cost estimate. Although a comparison between guideway configurations may be made using individual beam costs such as those developed in the previous section, this comparison may not show the least-cost guideway since the column and footing costs, which are directly affected by beam span length, are not considered. Therefore, the hypothetical scenario system is generated to serve as a baseline for comparison of various concepts. This 6 km system contains representative lengths of curved and straight as well as single and double lane guideway. Figure 63 shows typical straight beam concept costs. 1) right-of-way acquisition, 2) right-of-way preparation and 3) utility relocation are not considered in either the scenario or evaluation model analysis since these costs may vary radically depending on the guideway location. Cost estimates use 1978 dollars and average rates for major U.S. cities. Figure 66 indicates conventional guideway concepts summaries. Table 24 shows conventional guideway concepts scenario cost summary and Figure 68 shows conventional guideway beam span length versus cost.

Beam Shape	Concrete Double-Tee	Double Plate Girder	Double Plate Girder	Composite Wide Flange
Beam Shape Symbol				
Prefabricated Beam Span Length, m	18.5	24.5	24.5	24.5
Off-Site Costs, \$				
Materials	1848	7023	7023	8053
Labor	1042	3342	3342	1129
Variable Plant Overhead	866	2823	2823	820
Contingencies	298	518	518	309
Fixed Plant Overhead	417	1370	1370	728
Subtotal of Cost F.O.B. Plant	4592	15076	15076	8039
Site Costs, \$				
Shipping	580	335	208	180
Materials	863	1376	3464	2183
Labor	1643	2399	2027	3871
Equipment	1021	557	557	982
Contingencies	333	492	664	714
Direct Site Overhead	548	813	1095	1178
Overhead & Profit	842	1246	1680	1806
Subtotal of Site Cost	5611	7281	9695	11014
Total Beam Cost	10203	22294	24771	19053

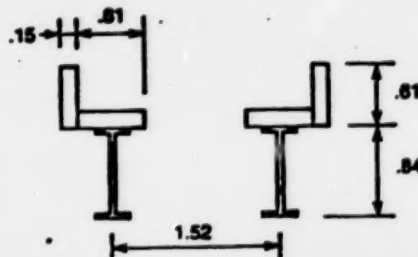
Figure 63. Typical Straight Beam Concept Costs



AREA (m ²)	.6586
WEIGHT (kN/m)	15.62
I (m ⁴)	.0552
S _T (m ³)	.1058
S _B (m ³)	.0793
No. Strd. Pre-ten.	14
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	18.5	10,203	15.0	9,596

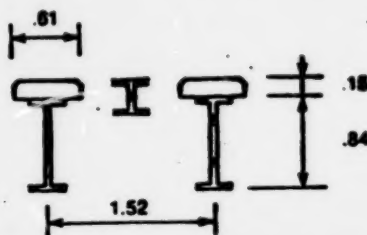
Guideway Scenario Cost - (1978\$) \$1,091/m
Concrete Double-Tee



AREA (m ²)	.2410
WEIGHT (kN/m)	13.86
I (m ⁴)	.0091
S _T (m ³)	.0242
S _B (m ³)	.0149
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	24.5	22,294	21.0	22,934

Guideway Scenario Cost - (1978\$) \$1,335/m
Double Plate Girder (Variation 1)

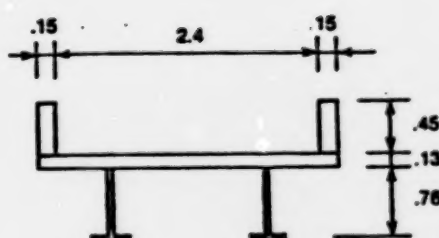


AREA (m ²)	.0713
WEIGHT (kN/m)	9.57
I (m ⁴)	.0091
S _T (m ³)	.0242
S _B (m ³)	.0149
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	24.5	24,771	21.0	25,482

Guideway Scenario Cost - (1978\$) \$1,439/m
Double Plate Girder (Variation 2)

Figure 66. Conventional Guideway Concepts Summary Sheets

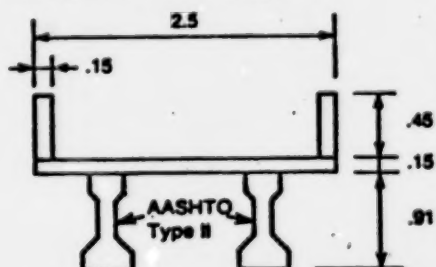


AREA (m ²)	.5124
WEIGHT (kN/m)	14.42
I (m ⁴)	.0082
S _T (m ³)	.0268
S _B (m ³)	.0141
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	24.5	19,053	21.0	19,598

Guideway Scenario Cost - (1978\$) \$1,198/m

Composite Wide Flange



AREA (m ²)	1.2325
WEIGHT (kN/m)	29.23
I (m ⁴)	.2994
S _T (m ³)	.3311
S _B (m ³)	.3570
No. Strd. Pre-ten.	44
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	20.0	13,902	18.5	13,304

Guideway Scenario Cost - (1978\$) \$1,164/m

AASHTO Girders



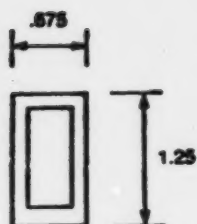
AREA (m ²)	.0275
WEIGHT (kN/m)	2.12
I (m ⁴)	.0023
S _T (m ³)	.0068
S _B (m ³)	.0068
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	18.0	8,589	15.0	8,295

Guideway Scenario Cost - (1978\$) \$908/m

Steel Girder

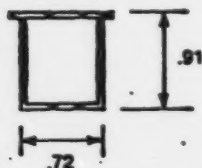
Figure 66. Conventional Guideway Concepts Summary Sheets (continued)



AREA (m ²)	.7325
WEIGHT (kN/m)	17.39
I (m ⁴)	.2171
S _T (m ³)	.2653
S _B (m ³)	.2176
No. Strd. Pre-ten.	28
No. Strd. Post-ten.	8

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	33.5	16,721	30.0	17,220

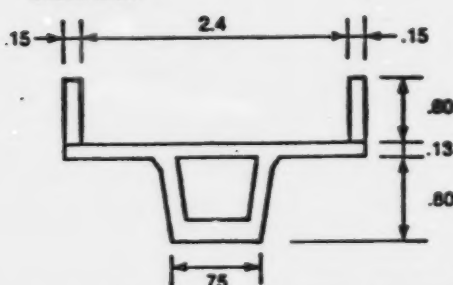
Guideway Scenario Cost - (1978\$) \$794/m
Concrete Box



AREA (m ²)	.0441
WEIGHT (kN/m)	3.40
I (m ⁴)	.0064
S _T (m ³)	.0156
S _B (m ³)	.0129
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	27.5	14,821	25.0	15,495

Guideway Scenario Cost - (1978\$) \$900/m
Steel Box

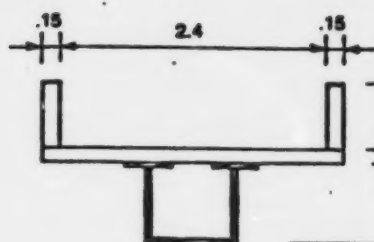


AREA (m ²)	.8510
WEIGHT (kN/m)	20.04
I (m ⁴)	.1377
S _T (m ³)	.1711
S _B (m ³)	.1900
No. Strd. Pre-ten.	36
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	27.5	20,177	24.0	20,779

Guideway Scenario Cost - (1978\$) \$1,103/m
Concrete Flanged Box

Figure 66. Conventional Guideway Concepts Summary Sheets (continued)

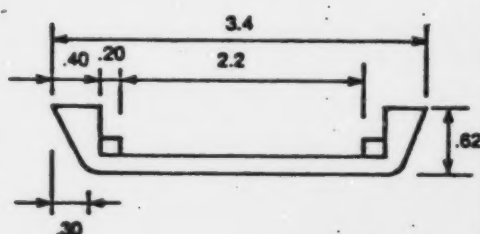


AREA (m ²)	.3648
WEIGHT (kN/m)	13.78
I (m ⁴)	.0388
S _T (m ³)	.0268
S _B (m ³)	.0153
No. Strd. Pre-ten.	0
No. Strd. Post-ten.	0

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	24.5	19,497	21.0	20,053

Guideway Scenario Cost - (1978\$) \$1,217/m

Composite Flanged Box

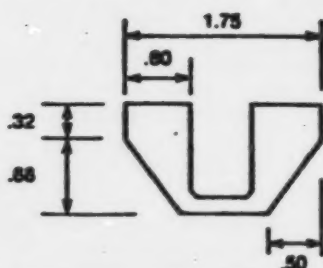


AREA (m ²)	.7000
WEIGHT (kN/m)	18.60
I (m ⁴)	.2047
S _T (m ³)	.0597
S _B (m ³)	.1197
No. Strd. Pre-ten.	30
No. Strd. Post-ten.	6

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	18.5	15,159	18.5	15,548

Guideway Scenario Cost - (1978\$) \$1,354/m

Concrete Channel



AREA (m ²)	.7885
WEIGHT (kN/m)	18.23
I (m ⁴)	.0387
S _T (m ³)	.1102
S _B (m ³)	.0904
No. Strd. Pre-ten.	32
No. Strd. Post-ten.	6

Vehicle Size	STRAIGHT BEAM		CURVED BEAM	
	Span (m)	Cost \$	Span (m)	Cost \$
Medium	22.5	16,983	20.0	17,381

Guideway Scenario Cost - (1978\$) \$1,197/m

Narrow Channel

Figure 66. Conventional Guideway Concepts Summary Sheets (concluded)

Table 24. Conventional Guideway Concepts Scenario Cost Summary

Item	Concrete Double Tee	Double Plate Girder (Variation 1)	Double Plate Girder (Variation 2)
Total Guideway Cost, \$	6,546,039	8,008,678	8,636,219
Cost/Lane-m, \$/lane-m	1,091	1,335	1,439
Beam Cost, \$	3,398,349	3,648,168	6,275,709
Percent of Total Cost	51.9	70.5	72.7
Column Cost, \$	1,057,000	793,000	793,000
Percent of Total Cost	16.1	9.9	9.2
Footing Cost, \$	2,090,690	1,567,510	1,567,510
Percent of Total Cost	31.9	19.6	18.2
Straight Beam Length, m	18.5	24.5	24.5
Straight Beam Cost, \$	10,203	22,294	24,771
Curve Beam Length, m	15.0	21.0	21.0
Curve Beam Cost, \$	9,596	22,934	25,482
Single-Lane Column Cost, \$	3,000	3,000	3,000
Single-Lane Footing Cost, \$	6,370	6,370	6,370
Dual-Lane Column Cost, \$	7,000	7,000	7,000
Dual-Lane Footing Cost, \$	10,710	10,710	10,710

Table 24. Conventional Guideway Concepts Scenario Cost Summary (continued)

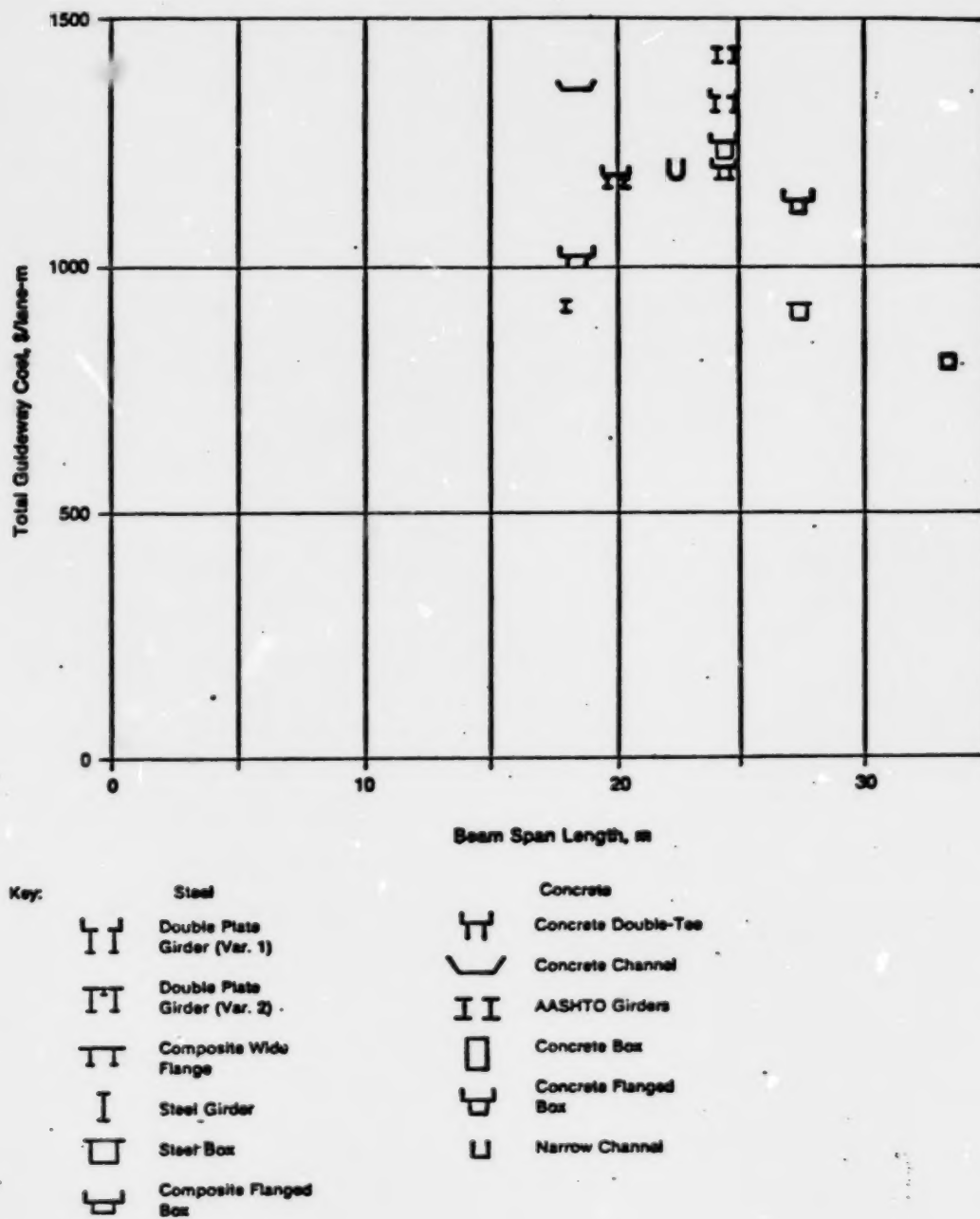
Item	Composite Wide Flange	AASHTO Girders	Steel Girder
Total Guideway Cost, \$	7,187,481	6,982,122	5,453,902
Cost/L-m, \$/lane-m	1,198	1,164	909
Beam Cost, \$	4,826,971	4,084,332	2,241,652
Percent of Total Cost	67.2	58.5	41.1
Column Cost, \$	793,000	973,000	1,079,000
Percent of Total Cost	11.0	13.9	19.8
Footing Cost, \$	1,567,510	1,924,790	2,133,250
Percent of Total Cost	21.8	27.6	39.1
Straight Beam Length, m	24.5	20	18.0
Straight Beam Cost, \$	19,053	13,092	6,569
Curve Beam Length, m	21.0	16.5	15.0
Curve Beam Cost, \$	19,598	13,304	6,295
Single-Lane Column Cost, \$	3,000	3,000	3,000
Single-Lane Footing Cost, \$	6,370	6,370	6,370
Dual-Lane Column Cost, \$	7,000	7,000	7,000
Dual-Lane Footing Cost, \$	10,710	10,710	10,710

Table 24. Conventional Guideway Concepts Scenario Cost Summary (continued)

Item	Concrete Box	Steel Box	Concrete Flanged Box
Total Guideway Cost, \$	4,763,169	5,399,722	6,616,023
Cost/Lane-m, \$/lane-m	794	900	1,103
Beam Cost, \$	3,059,689	3,317,222	4,524,153
Percent of Total Cost	64.2	61.4	68.4
Column Cost, \$	572,000	700,000	703,000
Percent of Total Cost	12.0	13.0	10.6
Footing Cost, \$	1,131,480	1,382,500	1,388,870
Percent of Total Cost	23.8	25.6	21.0
Straight Beam Length, m	33.5	27.5	27.5
Straight Beam Cost, \$	16,721	14,821	20,177
Curve Beam Length, m	30.0	25.0	24.0
Curve Beam Cost, \$	17,220	15,495	20,779
Single-Lane Column Cost, \$	3,000	3,000	3,000
Single-Lane Footing Cost, \$	6,370	6,370	6,370
Dual-Lane Column Cost, \$	7,000	7,000	7,000
Dual-Lane Footing Cost, \$	10,710	10,710	10,710

Table 24. Conventional Guideway Concepts Scenario Cost Summary (concluded)

Item	Composite Flanged Box	Concrete Channel	Narrow Channel
Total Guideway Cost, \$	7,299,886	8,124,099	7,183,748
Cost/Lane-m, \$/lane-m	1,217	1,354	1,197
Beam Cost, \$	4,939,376	5,040,969	4,637,898
Percent of Total Cost	67.7	62.0	64.6
Column Cost, \$	793,000	1,035,000	855,000
Percent of Total Cost	10.9	12.7	11.9
Footing Cost, \$	1,567,510	2,048,130	1,690,850
Percent of Total Cost	21.5	25.2	23.5
Straight Beam Length, m	24.5	18.5	22.5
Straight Beam Cost, \$	19,497	15,159	16,983
Curve Beam Length, m	21.0	16.5	20.0
Curve Beam Cost, \$	20,053	15,548	17,361
Single-Lane Column Cost, \$	3,000	3,000	3,000
Single-Lane Footing Cost, \$	6,370	6,370	6,370
Dual-Lane Column Cost, \$	7,000	7,000	7,000
Dual-Lane Footing Cost, \$	10,710	10,710	10,710



Report No. 3

Trip Report. Visit to modern American Transit systems.
ABAM November 26, 1980

A visit was made to the following transit systems:

- San Francisco
- Washington
- Baltimore
- Miami
- Atlanta

The purpose of the trip was to improve the background for the ICTS development program. The following information was discussed with the transit authorities visited:

- Trackwork component designs
- Trackwork construction procedures
- Trackwork maintenance
- Modifications made to initial design
- Organization and operation of maintenance departments

The following conclusions are made as a result of discussions of industry practices and experiences:

- Continuous welded rail
- Direct fixation trackwork
- Rails run continuously over aerial structures
- Field joining of rail
- Installation tolerances
- Rail wear and corrugation
- Direct fixation fasteners $+\frac{1}{4}$ " to $+1$ " lateral adjustment
- Direct fixation fasteners use bolted connections
- Rail installed using second pour techniques
- Up to $\frac{1}{4}$ " shims for final vertical adjustment
- AREA special trackwork
- Special connections at turnouts for thermal forces
- Hixson fastener for turnouts
- Switch machines base, level with base of rail
- Structural expansion joints kept out of special trackwork areas

Recommendations for ICTS guideway.

- Use continuous rail
- Rail expansion joints minimal
- Consider Hixson fastener
- Minimize loss of bolt preload
- Construction quality control
- Second pour installation method
- Standardize ICTS special trackwork geometrics
- Alternatives of switch/frog machines to minimize size of structure
- Hixson fastener type for special trackwork

Report No. 4Proposal for elevated steel guideway concepts for use in
ICTS. November 1975

This report has been prepared under the auspices of the Canadian Steel Industries Construction Council and Read-Harris Engineering.

Briefly, the report covers the preliminary engineering which has been carried out to evaluate several concepts. These concepts include several variations of steel box girders. These are rectangular, trapezoidal and semi-octagonal shapes. A truss type supporting structure fabricated using hollow structural sections is also presented.

First method - the deck is cast in place after the erection of the steel girders.

Second method - precast concrete deck segments can be fabricated in a precast plant. This would have several advantages:

Firstly, no forming would be required to support the deck as is the case with a cast in place deck.

Secondly, the quality control would be much improved in a plant.

Thirdly, savings in steel support structure weight could be realized.

Guideway Curves

In either the cast in place or the precast system, allowances can be made for curvature in the guideway.

The cast in place system allows for a more precise construction of curves, in that the forms can be built exactly to the desired curvature. Good quality precast units could be built with straight edges and the resulting chord lines at the edge of the deck would be aesthetically pleasing. Strike a balance between the acceptable

and desirable aesthetic standards in order to optimize the aesthetic acceptability vs. unit cost of the construction of the deck.

Guideway Girder Fabrication

The box girder is merely two plate girders in which the bottom - flanges have been widened sufficiently to form one flange.

For the fabrication of straight line girders, the fabrication and utilization of materials can be very efficient. Flange plates are cut from large steel plates supplied from the steel mill. Little or no waste material results from this cutting.

Girder Concepts

No. 1 - Type A girder is the most efficient in terms of fabrication. This type of box is a pleasing, clean lined structure. Girder B is a trapezoidal box section with reduced bottom flange plate width. This allows for more efficient use of flange plate material. Type C is a slight variation from Type B in that the web is not just sloping at one rate but a break is introduced to the web for aesthetic reasons.

No. 2 - The use of tubular or hollow structural sections has become more evident (Plate 4).

No.3 - The girder type shown on Plate No. 5 should be fabricated at prices comparable to box girders. The girder shape would be created by bending the plate at 3 different points to form the box.

Curved Guideways

Generally, the direction change could be accommodated by fabricating curved girders but this would result in significant waste material. Further study should be given to the possible use of straight chord sections of about 26' length which would then be welded together to form the curve.

Guideway Piers

Concrete piers are generally used for bridges and this could be an acceptable material for this guideway system. All piers could be precast. It is felt, though, that further study should be carried out in the use of steel piers.

Guideway System Corrosion Protection

First method would be to apply a protective coating to the structure. This coating will have a life expectancy of 20 years.

Alternative is to use atmospheric corrosion resistant steel. The metal will develop an oxide which is uniform dark brown to purple black colour. While weathering is taking place, any concrete surfaces would have to be protected to prevent staining. This problem would be minimized if steel piers were used.

Guideway Costs

Tables 6 and 7 summarize the total construction costs. These costs must be increased by \$12 per foot to allow for protective coating if this is selected as the method of protection.

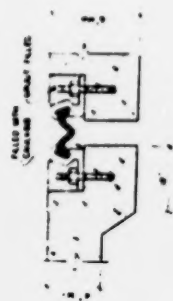
Conclusion

Most pleasing structure would be Concept No. 3 and Concept No. 1A, in that order.

The piers for the system should be considered using steel.

Weathering steel should also be used in its natural state and proper attention given to surrounding areas to minimize staining.

Elevation of Typical Structure



TYPICAL EXPANSION JOINT



CAST IN PLACE



PRE-CAST

ERECTION PROCEDURES

STEEL GUIDEWAY CONCEPTS

CONSTRUCTION SYSTEMS

Prepared for

WILSON, RASMAN, and STOLBERG, WCP

SPONSORED BY:
Canadian Steel Industries Construction Council

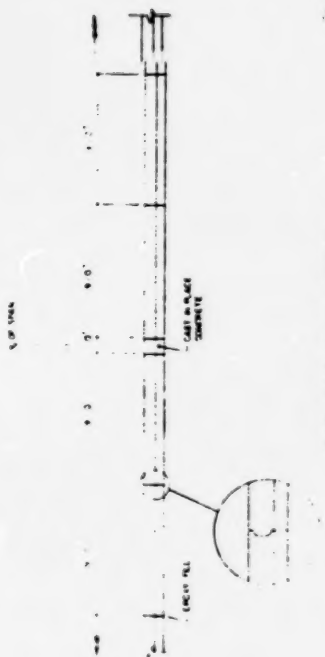
052

Need-Base Engineering

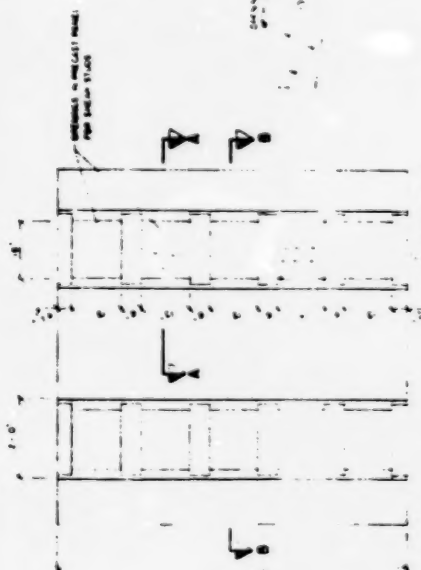
50



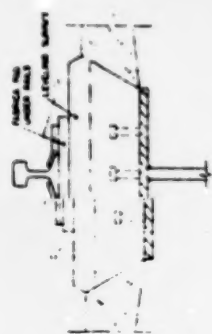
SECTION B-B
PRE-CAST ALTERNATE DECK
SCALE 1/4" = 1'-0"



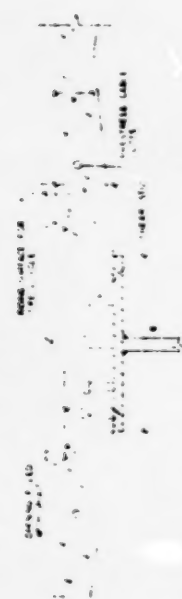
PRE-CAST SLAB JOINTS
SCALE 1/4" = 1'-0"



PLAN OF PRE-CAST CONCRETE
PANEL
SCALE 1/4" = 1'-0"

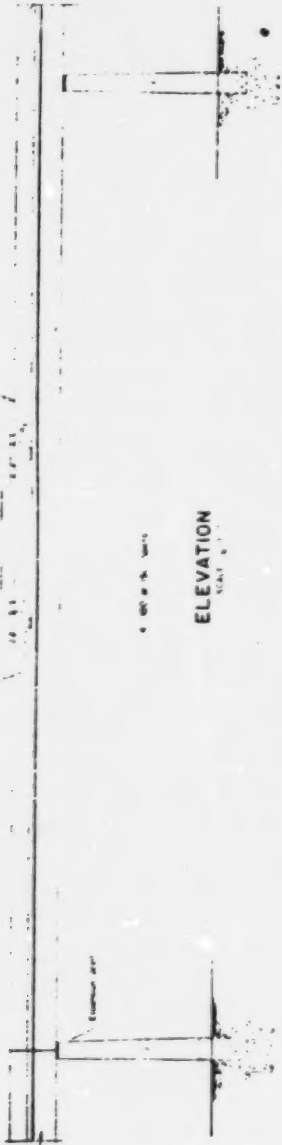
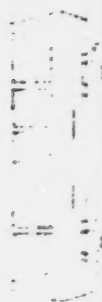


FLANGE - DECK DETAIL
STEEL WHEEL-RAIL
SCALE 1/4" = 1'-0"



SECTION A-A
FLANGE - DECK DETAIL
RUBBER TIRE VEHICLE
SCALE 1/4" = 1'-0"

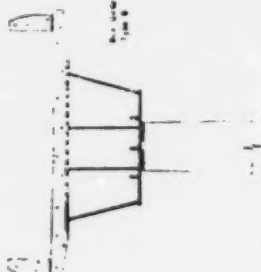
SPONSORED BY: Canadian Steel Industries Construction Council		51
PRE-CAST DECK DETAILS		Prepared for: Road - Harrie Engineering



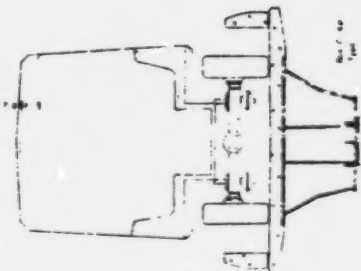
ELEVATION
SCALE 1/4" = 1'-0"



DOUBLE TRACK SECTION
SCALE 1/4" = 1'-0"



TRAPEZOIDAL BOX SECTION
SCALE 1/4" = 1'-0"



FOLDED WEB BOX
SCALE 1/4" = 1'-0"



RECTANGULAR BOX SECTION
SCALE 1/4" = 1'-0"

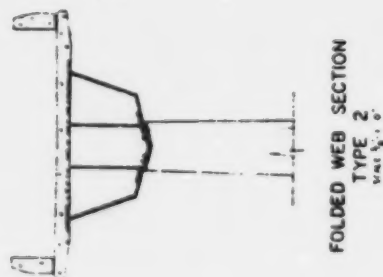
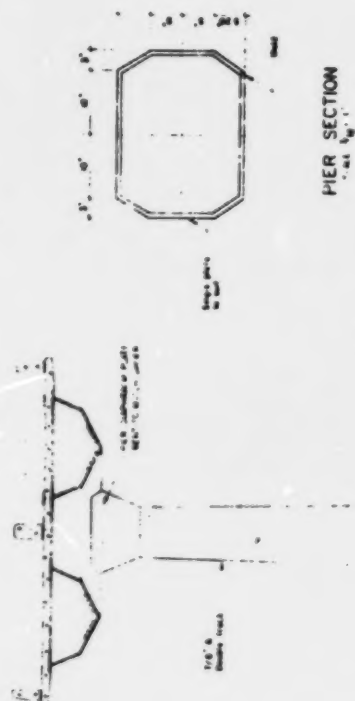
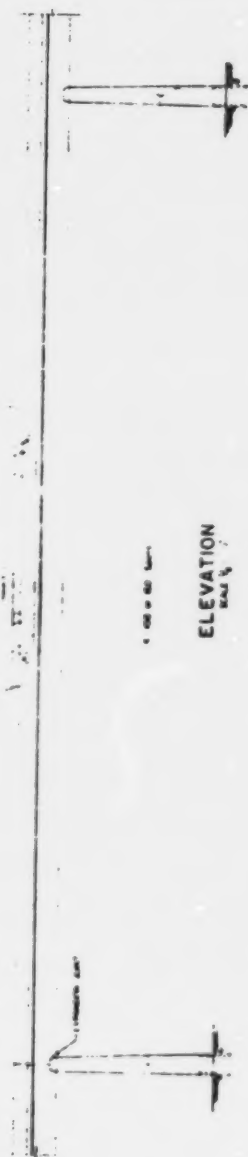
STEEL GUIDEWAY CONCEPTS

CONCEPT No. 1
Proposed for

SPONSORED BY
Canadian Steel Industries Construction Council

Read-Harris Engineering

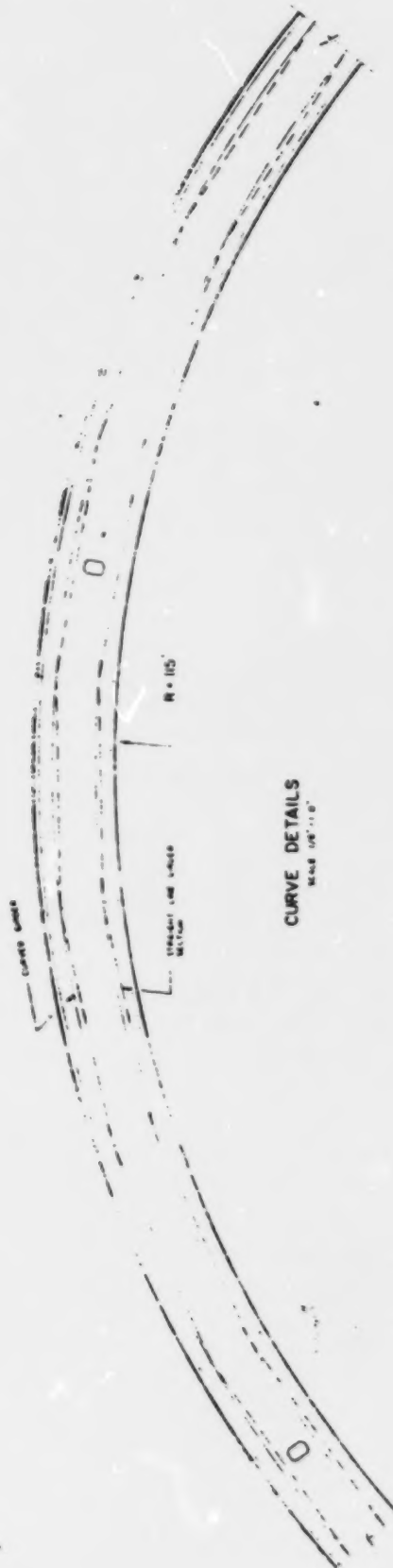






STATION ENTRANCE FOR ELEVATED GUIDEWAY

SCALE 1/8" = 1'-0"



CURVE DETAILS

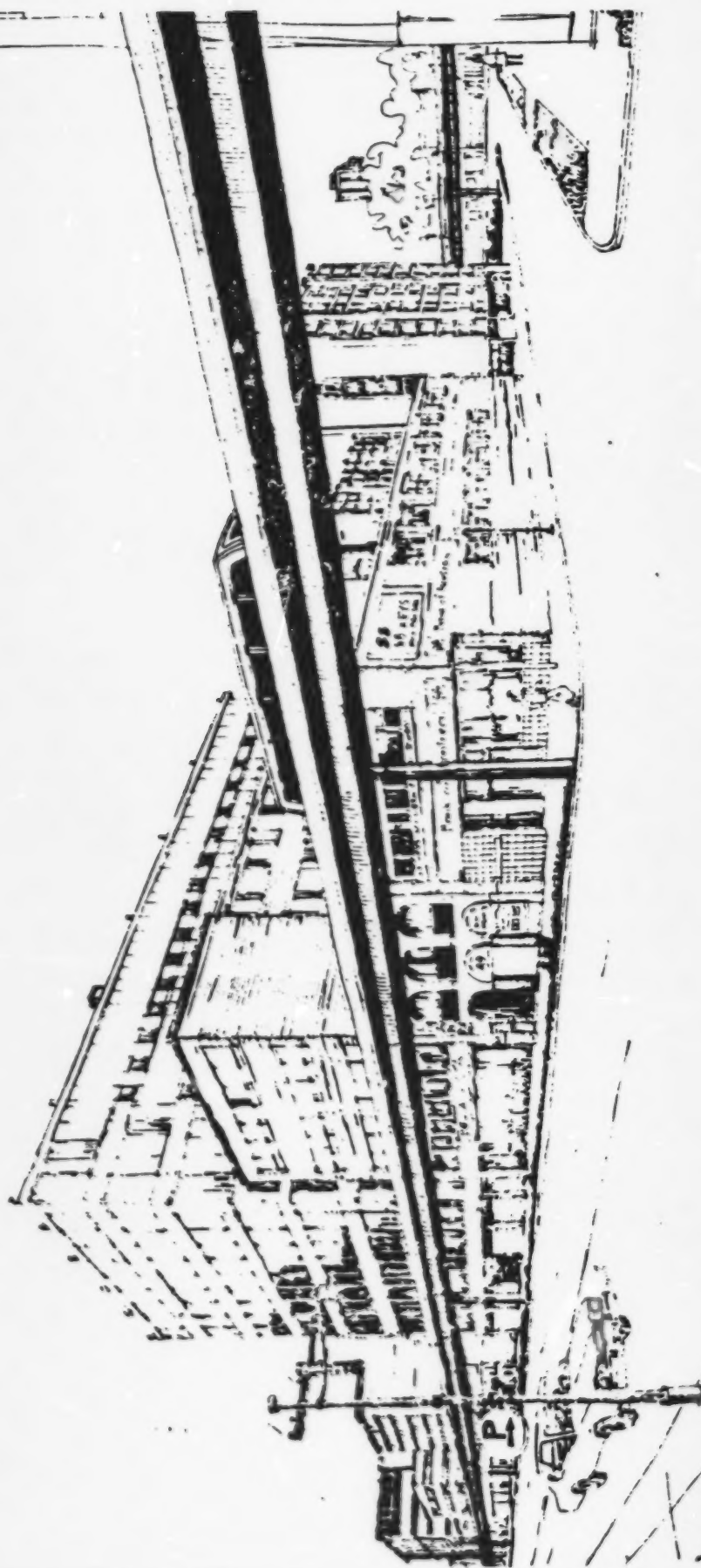
SCALE 1/8" = 1'-0"

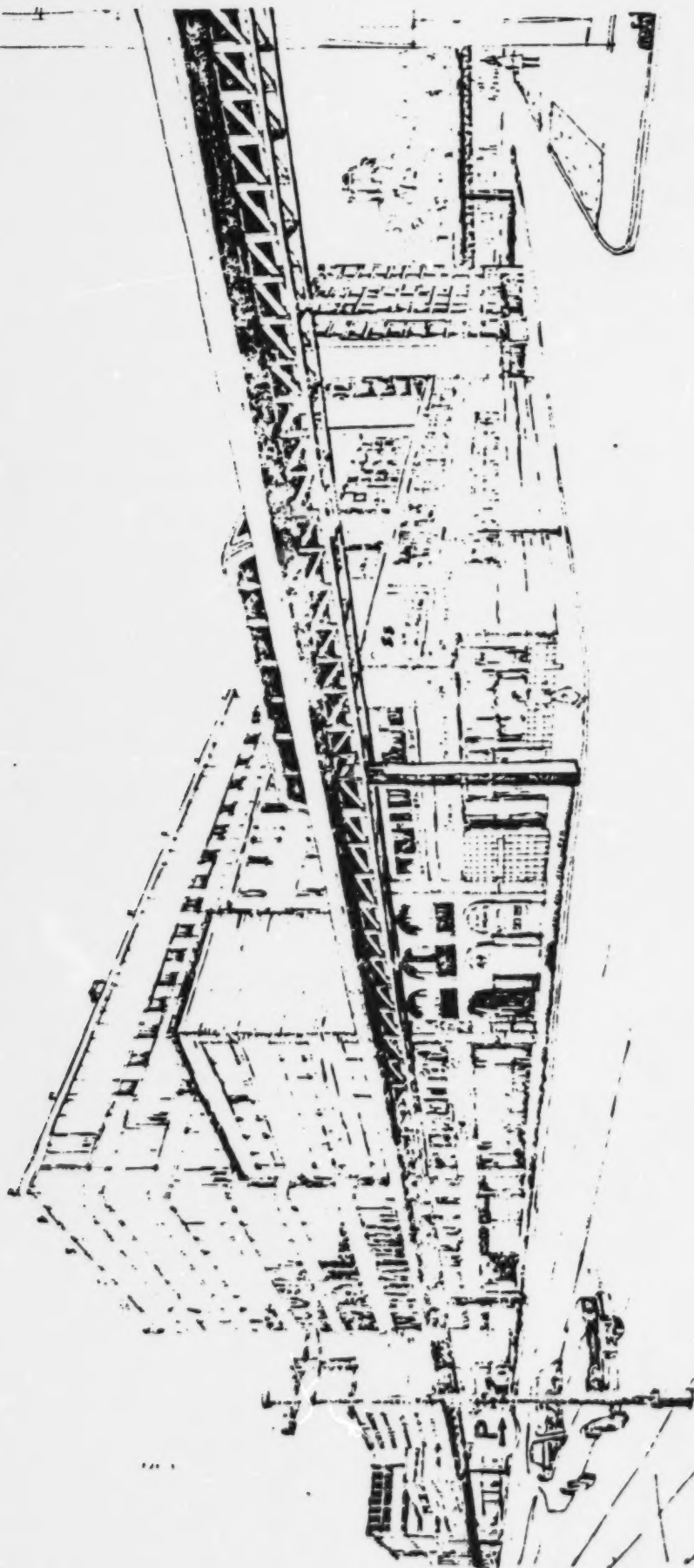
PROPOSED CURVE & SWITCHING
DETAILS

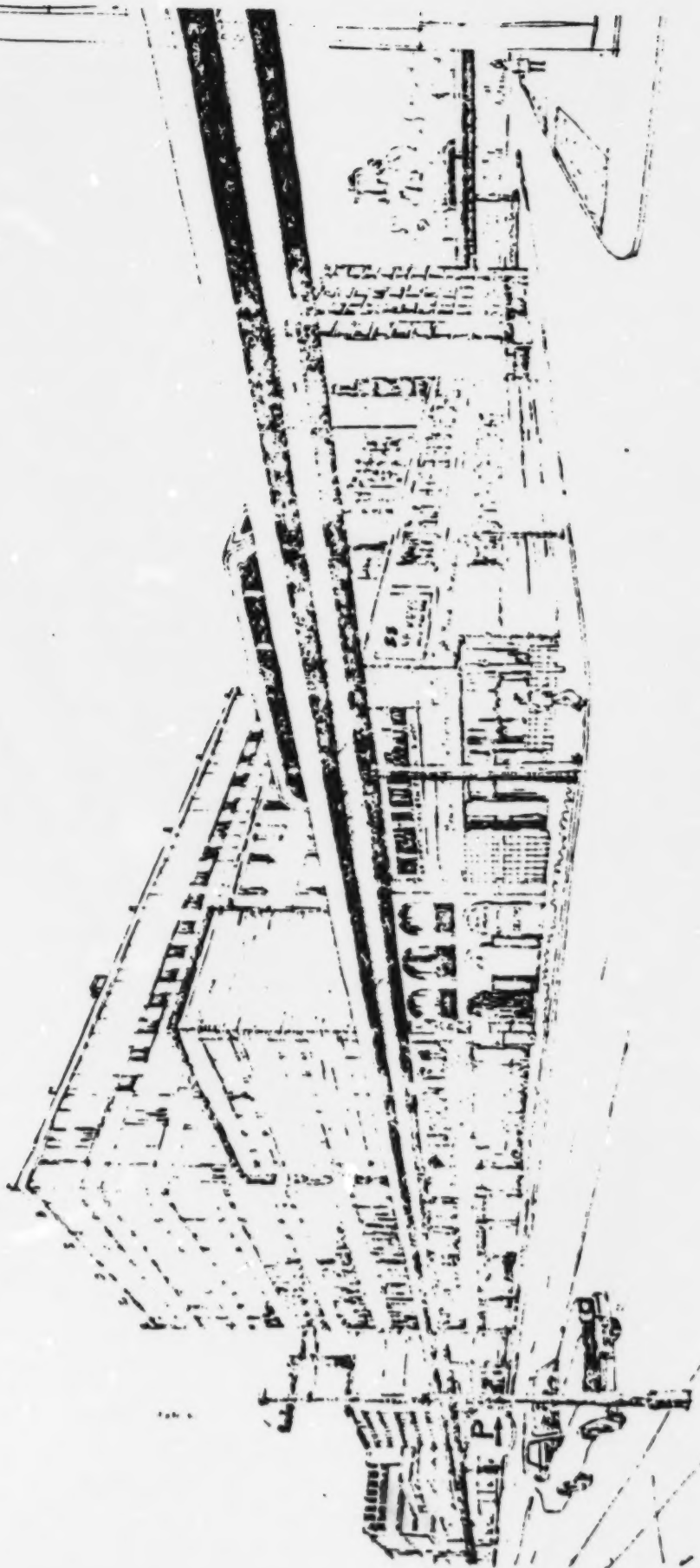
SPONSORED BY
London Steel Industries Construction Council

Head-Office Engineering

54







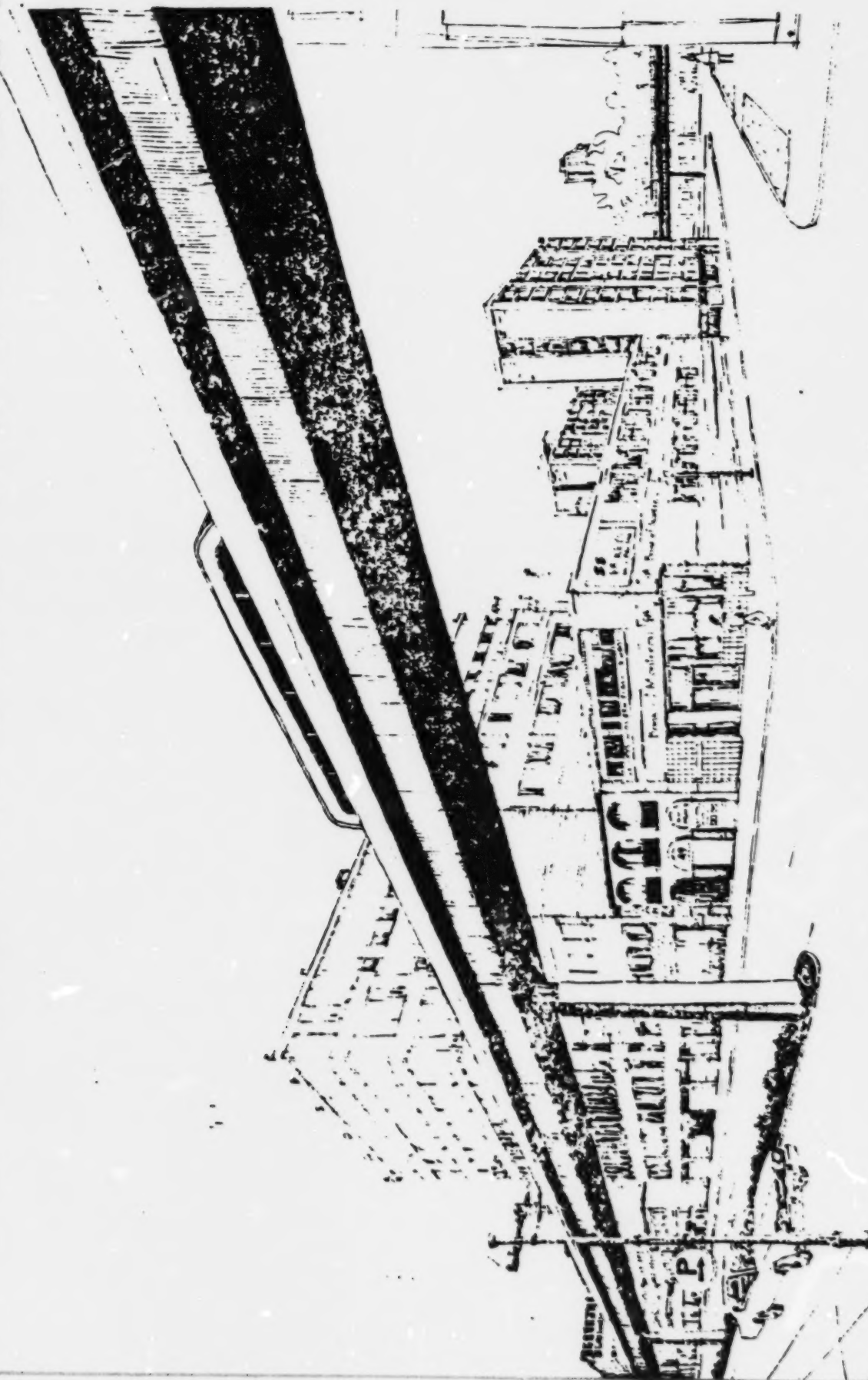


TABLE 9

SUSPENDED

CONCEPT 1

CONCEPT 2

CONCEPT 3

	150' Spans		100' Spans		150' Spans		100' Spans		150' Spans		100' Spans		150' Spans		Deck		Pier		
	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	Wt/Ft	\$/Ton	\$/Ft	\$/Ft ²		Each	\$/Ft
Supplier 1																			
Girder A	206	1,248	129															75	\$257.
				391	934	183.												50	\$286.
Girder B	206	1,248	129															75	\$257.
				391	894	175.												50	\$278.
Girder C	206	1,289	133															75	\$261.
				391	934	183.												50	\$286.
							220	1,620	178.						Typical All System		Typical All System	75	\$306.
							280	1,297	182.						System 5.75	53.	7500.	75	\$310.
										404	1,065	215.						50	\$318.
										404	1,169	236.						75	\$269.
Supplier 2															462	1,014	234.	50	\$287.
Girder A	206	875	90															75	\$218.
				391	875	171.												50	\$274.
Girder B	206	875	90															75	\$218.
				391	875	171.												50	\$274.
Girder C	206	875	90															75	\$218.
				391	875	171.												50	\$274.
							220	875	96.						Typical All System		Typical All System	75	\$224.
							280	875	123.						System 5.75	53.	7500.	75	\$251.
										404	875	177						50	\$280.
										404	875	177.						75	\$230.
																		50	\$252.

Report No. 5Elevated Structures for Mass Transit Systems - American Iron and Steel Institute

Much has been written about innovative guided transit systems. However, no single reference has brought together the various facets of elevated guideway design and construction for such new systems.

This book is aimed primarily at the structural engineer faced with the task of designing an elevated guideway. The emphasis is on aspects which distinguish transit guideways from highway and railway bridges.

Design Methods

1. Working Stress
 2. Load Factor
- Basic theory and definitions
Vehicle simulation
Guideway simulation
Frequency components of vibration
Separate guideway and vehicle analysis
Random Vibrations

Types of Construction

An alternative to twin I-girders is a box girder. Box girders are frequently desired for their pleasing appearance. The webs may be vertical or inclined.

Various types of grades of steel are available for use in guideway construction. It will usually be economical to use the higher strength when such usage results in less weight. Atmospheric corrosion resistant steel can be used to eliminate initial and future painting.

Guideways consisting of simple spans are not recommended where there is any difficulty in achieving ride quality. Simple spans can prove advantageous for speeding erection and minimizing field

work.

Continuous span units have substantial advantages over simple spans relative to vehicle vibrations. Another possible approach for guideway construction is to design and construct the units as simple spans for dead load and as continuous spans for live load. Slabs can be cast on the simple spans either in place or prior to erection provided suitable splice details are developed. Field splices are required in continuous units to facilitate shipping.

Selection of span length is affected by many considerations - crossing of streets and other physical obstacles - general appearance of structure is a factor of pier height and span length - transportation and erection.

When other constraints leave room for choice, economy is the governing factor.

Three methods of fabricating a curved girder - cutting the flanges to the curve and welding the web to the flanges in the curved configuration - heat curving a girder initially fabricated straight and - cold bending a straight girder. The guideway cross section on a curved alignment is affected by the superelevation of the slab. There are a number of ways that superelevation can be achieved. Twist the cross section - vertical curvature superimposed on horizontal is complex - rotate the slab and keep girders vertical but offset by amount of slope, appearance could be less attractive.

Tolerances and deviations - limitations and corrective measures have to be considered.

Report No. 6

Hawker Siddeley Canada Limited
Technical Design Proposal for Intermediate Capacity Transit
Demonstration System, Volume III, Technical Design Proposal
(Civil) Prepared by Cole, Sherman & Associates Ltd., Consultants

The feature of the system which has the major impact on guideway design is the single-sided linear induction motor. The use of this motor enables the deck of the guideway to be entirely free of obstructions in all sections except at switches. This is a major advantage when considering the problems of snow and ice removal, safety, flexibility, maintenance and switching.

With the exception of special locations, where site conditions dictate span requirements, the guideway design is subject to two basic philosophies:

- (i) Long span, deep beam (Proposal A)
- (ii) Short span, shallow beam (Proposal B)

This proposal offers a guideway system for each of these philosophies as applied to the C.N.E. site with cost estimates and other information necessary to make a choice.

The approach taken to the guideway design was first to consider the various alternative beam sections, materials and construction methods available for guideway application. Applying this approach the choice of guideway sections was narrowed to basic beams for which span/cost ratio curves were plotted, leading to optimum alternatives for varied applications.

SELECTION FACTORS

In addition to the design factors listed in Section 2.3 there are many other considerations that led to the selection of the optimum alternatives.

Factors considered in the comparative evaluation include:

- Economy
- Aesthetics
- Geometry of the alignment and problems at switches
- Restraints on span lengths imposed by existing roads and buildings

- Familiarity of the construction industry with the construction of the various segments of the guideway and also the present and anticipated competitive condition of the construction market.
- Availability of specialized local construction materials.
- Speed and ease of construction.
- Schedule restraints - i.e. winter conditions and restricted operations during C.N.E. and Ontario Place operating seasons.
- Maintenance costs of the guideway during normal life span (A4.3.1 (vi)).
- Noise generation of the system such as resonance in structural members (A3.4.3).
- Freedom from interference with automatic vehicle control system.
- Ability to adapt to the future use of prefabricated structural components on an expanded system.
- Ability to adapt to other systems such as air supported or magnetically levitated vehicles.
- Ability to meet required dimensional tolerances in the riding surface, reaction rails and switch rails.
- Ease of snow removal by other than surface melting techniques.
- Facility to incorporate a heating system in the deck.
- Facility to enable construction of the many changing rates of curvature and superelevation.

The structural steel industry, the precast concrete industry and general contractors were consulted. The complexity of this project and the required tolerances imposed more stringent requirements than the local construction industry generally encounter in bridge structures.

A compromise solution was needed between the best features of each whilst considering future application of each alternative for an expanded system.

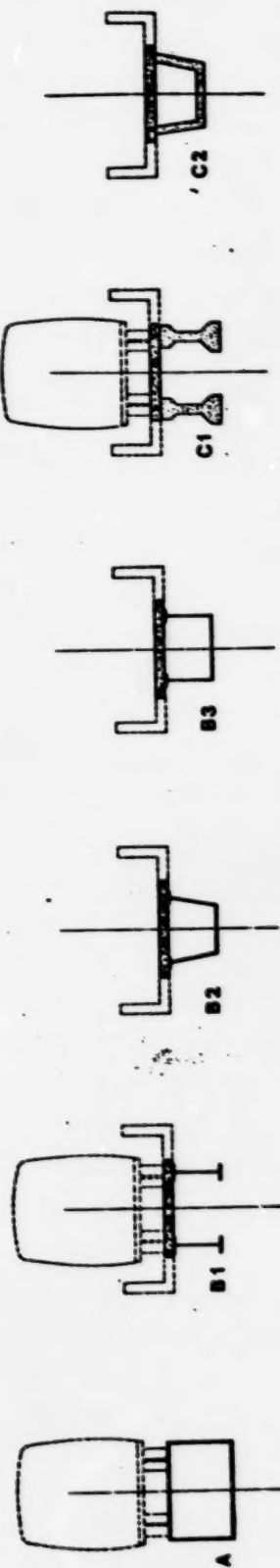
Possible structural concepts ranging from simple to the more sophisticated, each with the ability to satisfy the requirements as shown in Figure 2.15, are discussed.

STRUCTURAL STEEL

Alternative A - Box Section

This is the simplest solution in terms of meeting the geometric requirements and varying span lengths, by shop fabrication from steel plates.

2.15 Possible Structural Concepts - Structural Steel and Prestressed Concrete.



It has merit in speed and ease of construction. Use of atmospheric corrosion type steel leads to favourable maintenance costs, except for staining of sub-structure which must either be corrected in the future, or prevented by application of a special compound to the concrete pier surfaces.

A serious disadvantage is the probability of vehicle activated vibrations generating and propagating resonance in the girders. Countering this phenomenon by adding stiffeners or using thicker plates would escalate the cost.

Control system interference problems are a major concern in any structural steel guideway.

Custom fabrication to develop curved and superelevated members is costly. Field correction or adjustment of any misalignment is difficult.

This steel box section is considered to have poor visual appeal, particularly in the early years of weathering.

Alternative B1 - Double I-Beam with Cast-in-Place Concrete Deck
Alternative B2 - Open Box Girder, Sloped Sides - Cast-in-Place
Concrete Deck

Of the structural steel concepts this has the greatest aesthetic appeal. It is structurally efficient due to the narrow bottom flange. It has the same advantage as the previous section except that the cost escalates dramatically when combined curved and superelevated members have to be fabricated.

This section is suited to larger span lengths on straight stretches of guideway (in the order of 120 feet) and its future application could be considered.

Alternative B3 - Open Box Girder, Vertical Sides, Cast-in-Place Concrete Deck

The advantages and disadvantages applicable to Alternatives B1 and B2 also apply to this solution, but with the following modifications:

It has less aesthetic appeal than the sloping-side box although construction costs will be less in curved alignments.

The straight-sided box has been considered for this study and a cost curve for different span lengths has been incorporated in the comparative cost curve diagram. Figure 2.17.

For aesthetic and cost reasons, but primarily due to anticipated vehicle control system interference problems, the structural steel alternatives are not suitable for the present applications.

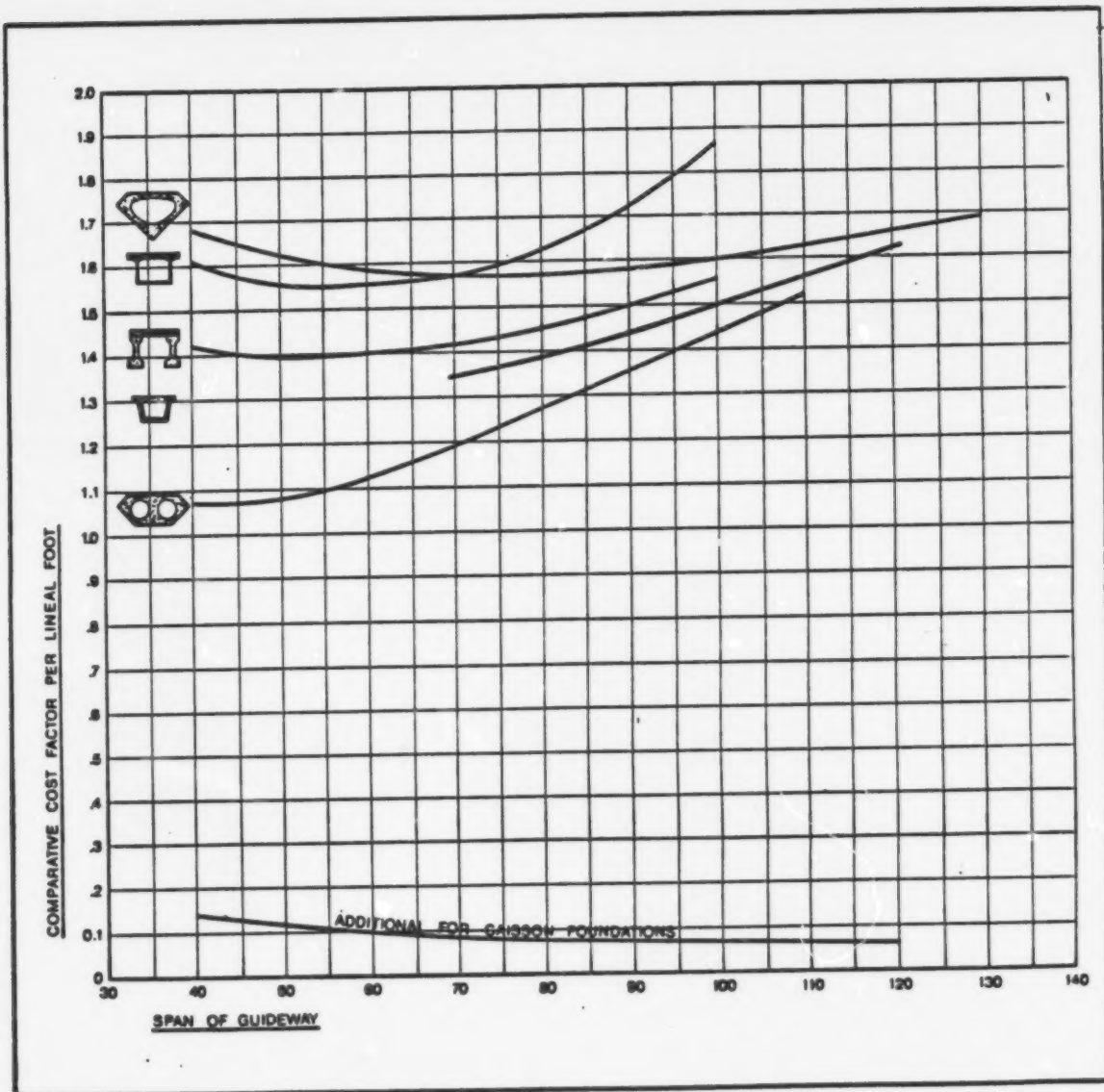
Precast Concrete

Two major precast manufacturers conceded that all switch sections would require to be cast-in-place concrete. Twelve switches presently planned for the guideway will be supplemented with additional switches for take-off to an expanded system. This factor combined with a number of irregular span lengths, leaves few applications for standardization suited to factory mass production. Also, when mixing the methods, it is feared that matching of colour and surface would be difficult. Differential long term weathering between the two may develop an unsightly structure in the future.

Development and fabrication of such formwork is costly. A further constraint is imposed by the curvature of the elements which would restrict the lengths that can be shipped. This would entail construction of temporary falsework bents and segmental construction.

This could be a viable solution if long tangent sections were the rule rather than the exception. Because of the many disadvantages of the precast concrete method when applied to the

2.17 Guideway Alternatives. Comparative Cost Curve Diagram.



demonstration system this method is eliminated from present consideration.

Selected Construction Method, Cast-in-Place Concrete

The merits of cast-in-place concrete construction are found in its suitability for building to any curvature, superelevation and varying width (as at switches), by adjusting the formwork under precise engineering control on site. Lead time is minimal; after tender award a contractor can quickly become mobilized. Speed and ease of construction are assured.

The competitiveness of the general construction industry is keen for this kind of work and market conditions are anticipated to be favourable, thus promising the best possible price.

Cast-in-place structures will require proper engineering of falsework to accommodate existing traffic routes. As this is common practice in highway construction today, it is not considered an impediment. The narrow width of the guideway ensures speedy falsework erection and relocation.

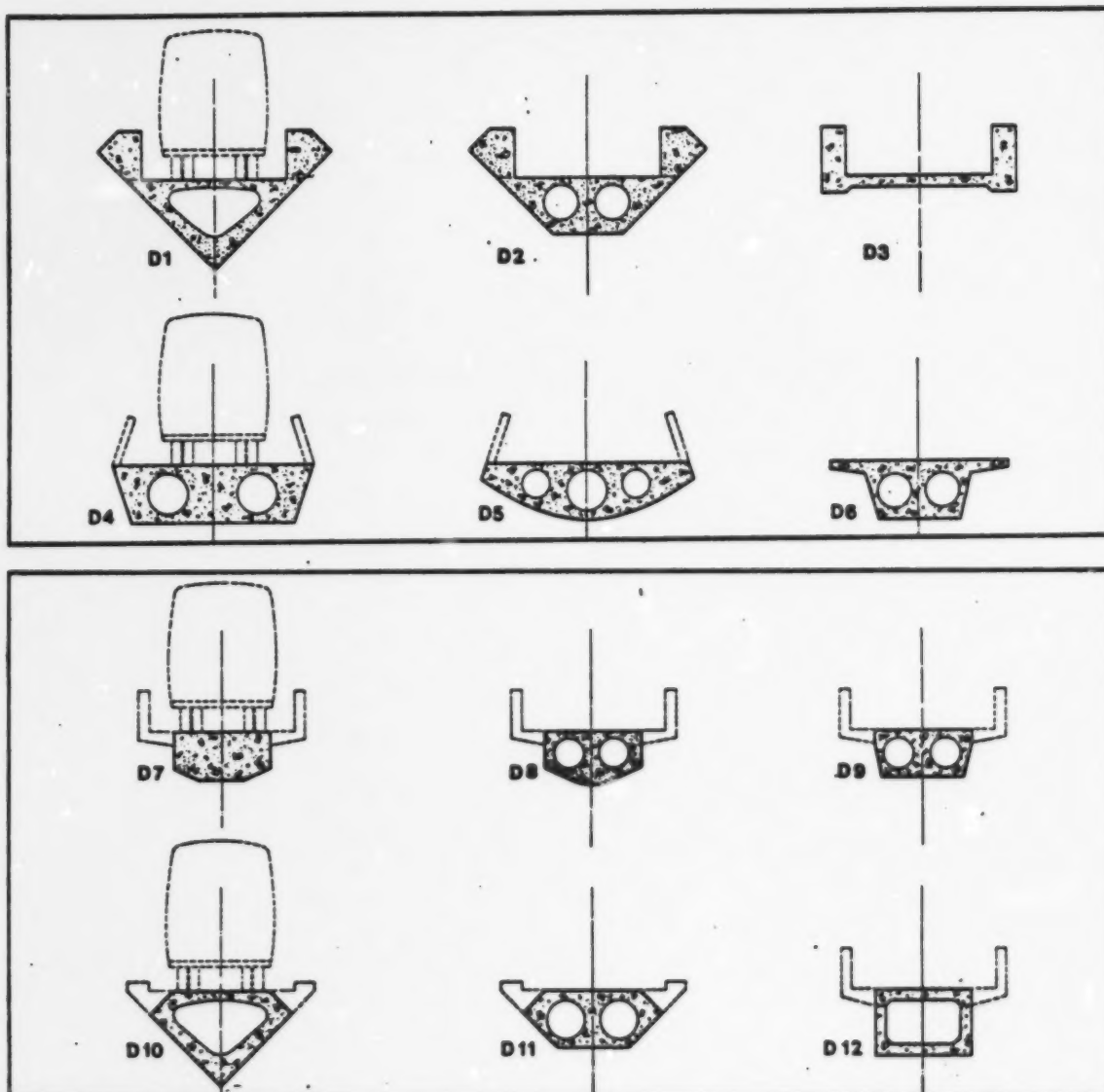
Cast-in-place construction is advantageous for the casting-in of anchor bolts, electrical conduits and inserts requiring firm anchorage.

A prime consideration in determining the shape of the guideway is that of aesthetics.

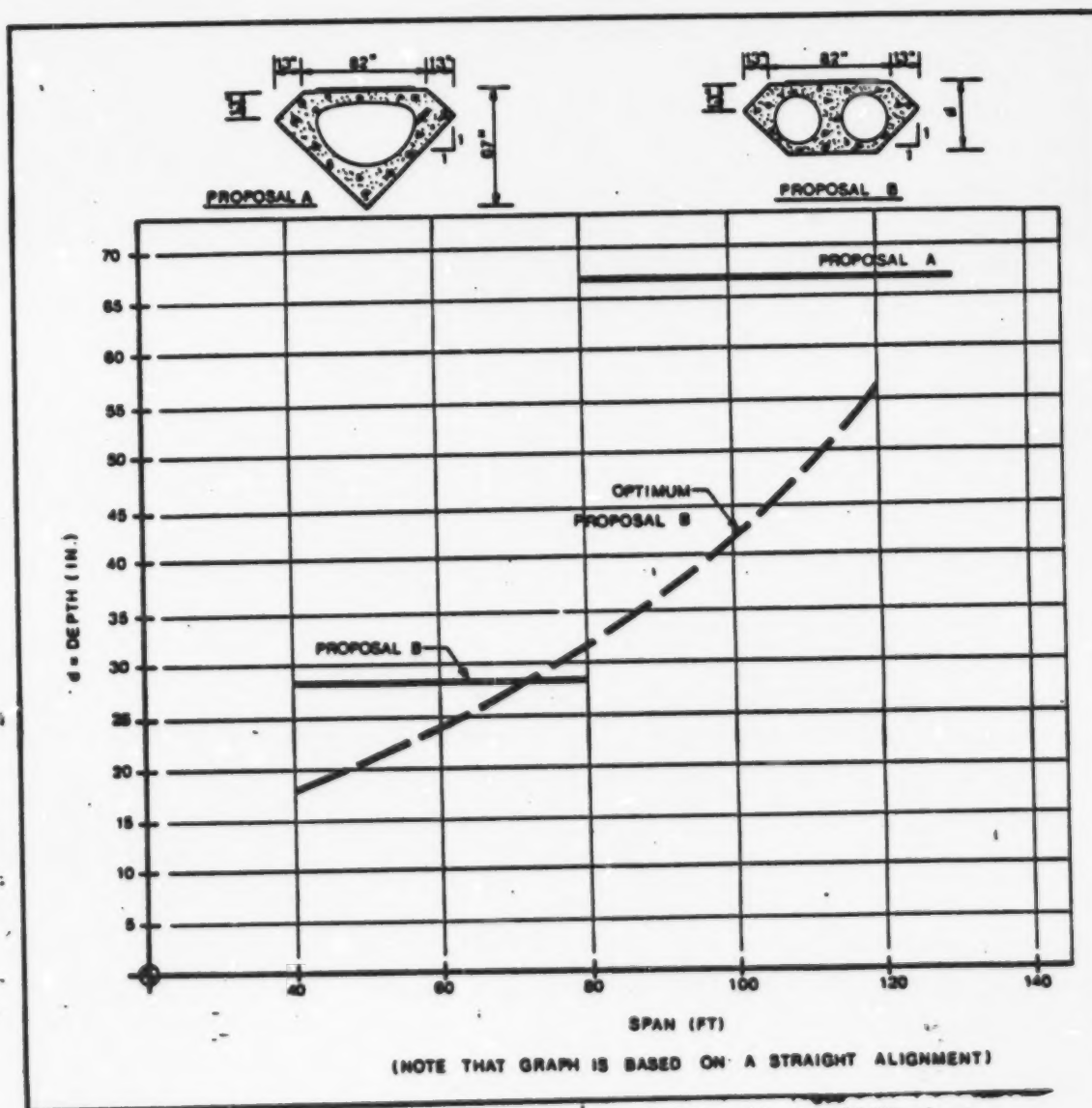
The aesthetically preferred shape 10, basic V-section, and the modification, shape 11 (Figure 2.16) are the final shapes selected to provide the best combination of aesthetic and economic qualities as dictated by the combined requirements of the vehicle and the site.

Thus we have a beam 67" deep of sufficient strength to span 130 feet. A relationship between the depth of the section and the span length is shown in Figure 2.21.

2.16 Possible Structural Concepts. Cast-in-Place Concrete.



2.21 Cross Sections and Optimum Span Ranges for Proposals A and B.



It is realized that not only the overall, but also the detailed, appearance of the guideway is important. It is also recognized that, when concrete is selected as the main building material, the surfaces should be homogeneous. Ideally the surface finish should look similar to that of precast concrete. To accomplish this it is recommended that either steel, high-density plywood, or fibreglass-coated forms be used, thus assuring not only a smooth surface, but also associated economics due to the many re-uses that these forms have. The forms can be held up by steel tubing type falsework that is vertically adjustable as shown in Figure 2.22.

Piers

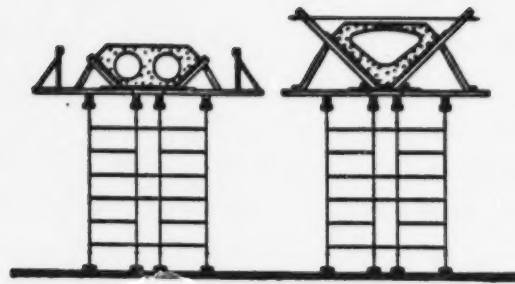
The reinforced concrete circular section is the most effective and economical member for this application and is therefore selected. The piers are carried below grade to the footing or pile cap. The guideway is supported on the piers by stainless steel-on-teflon pot bearings suited to the specific requirement at each column location. These have a proven record and perform well in cold climatic conditions, allowing for the large accumulated expansions associated with long continuous structures.

Deck Topping and Finish

It is recognized that the surface of the poured guide beam will have difficulty meeting the desired requirement. A concrete topping with a hardened finish of at least 3" thickness will be applied over the poured concrete surface to the precise finished elevations.

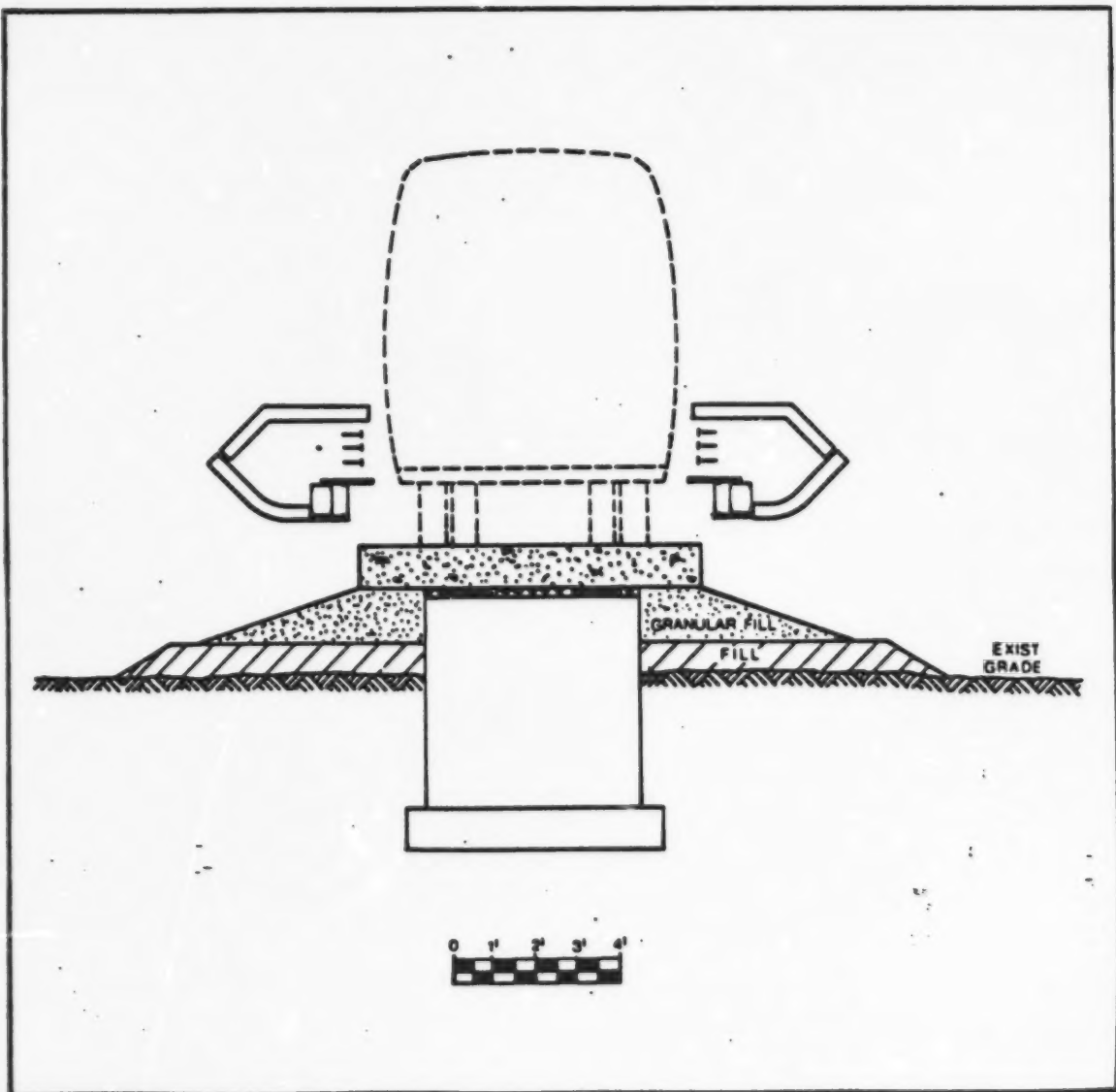
Atmospheric corrosion-resisting structural steel conforming to CSA G40.11 1969 specifications will be used for all structural steel members.

Most past examples of elevated transportation systems are massive and overbearing and totally insensitive to the environment. The form and visual effect of the elevated structures will have a



2.22 Framework Arrangements.

2.23 Guideway at Grade.



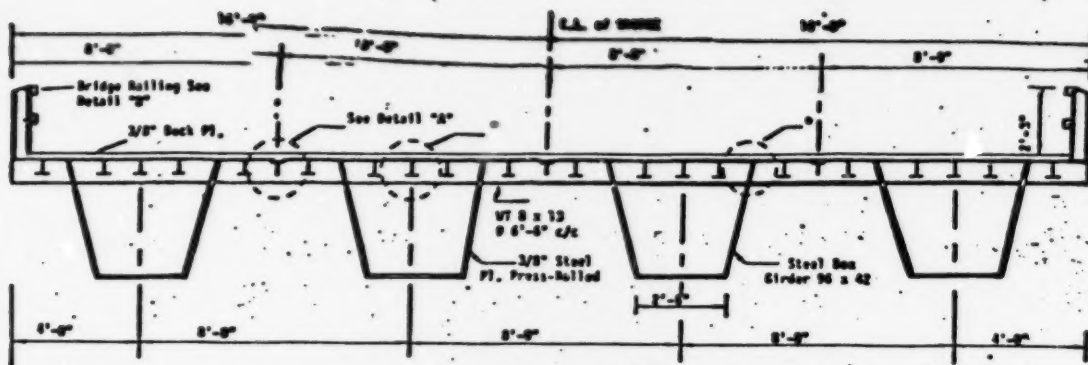
fundamental impact on the environment. The triangular shape eliminates the most objectionable part of any elevated structure - the flat underside which is always dark and reads like a black roof above the pedestrians on the ground. Because of this it will appear lighter and less oppressive when viewed from below. The triangle also allows optimum sun penetration because of its angled planes. These two considerations are perhaps the most important visual considerations in terms of public acceptance.

Report No. 7Prefabricated Press-Formed Steel
T-Box Girder Bridge System
ASC Journal Third Quarter 1979

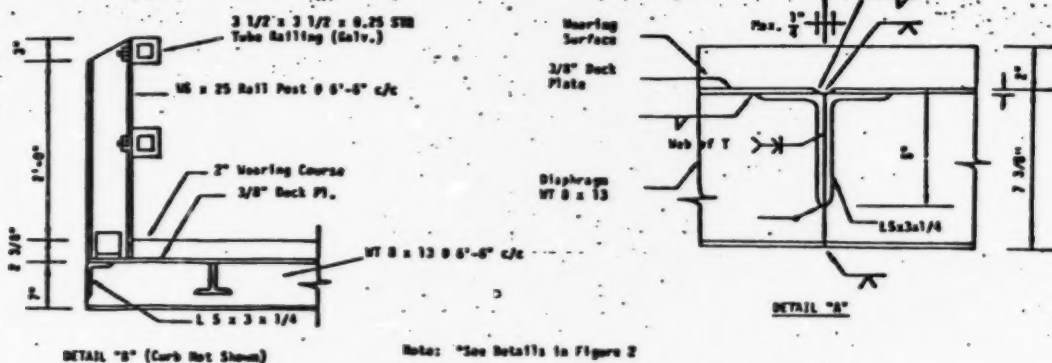
The conventional bridge construction methods are time-consuming and costly because of the high costs of material and labour. In addition, bridge maintenance costs are increasing rapidly. Hence, the need for more economical techniques of rehabilitating existing bridges and building new ones is well recognized. This paper presents the details of an all-steel superstructure system that is economical and suitable for span ranges of 40 to 100 ft. It is referred to as the press-formed prefabricated steel box girder bridge system.

The proposed system (Fig.1) consists of a trapezoidal trough section which is press-formed from 3/8-in. thick A36 steel plate and shop-welded to a 3/8-in. thick steel top flange with 6-ft. or 8-ft. widths. The top flange of the proposed girder is stiffened by a system of longitudinal ribs and partial depth intermediate diaphragms. The webs and the bottom flanges are not stiffened internally or externally. The ends of the girders are closed by a 3/8-in. thick plate diaphragm welded all around the flanges and webs of the girder. In addition, bearing stiffeners have also been provided. Typical girder sections are shown in Figs. 2 and 4. Different girder sections (Figs. 2 and 3) are suggested for different span lengths.

Two functional considerations for economy in bridge design are: (a) efficient utilization of the material by optimizing the dead weight of the structure and (b) minimizing the shop and field operations required to fabricate and erect the structure. Innovative application of high-strength steels and welding techniques have considerably enhanced the efficiency (strength-to-weight ratio) of steel structural members in recent years. Generally, the dead weight savings in superstructure are much more significant and important for long-span bridges. However, in comparison with the present practice for the short-span bridge



SECTION THRU BRIDGE AT MIDSPAN



DETAIL "B" (Curb Not Shown)

Notes: "See Details in Figure 2"

Fig. 1. Typical bridge section

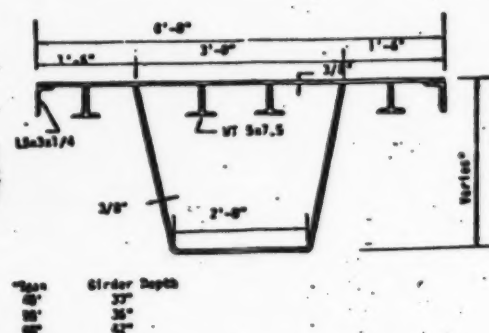
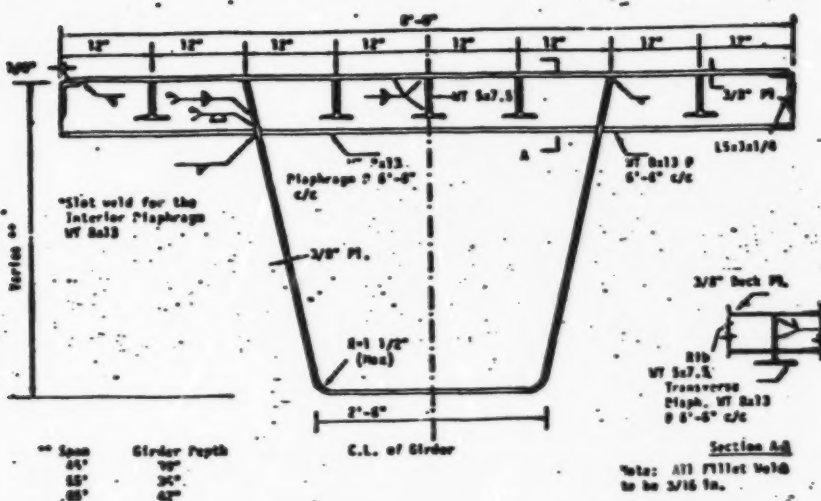


Fig. 2. Girder section showing fabrication details (8 ft wide) for 65-ft span

Fig. 3. Girder section (6 ft wide) for 45-ft span

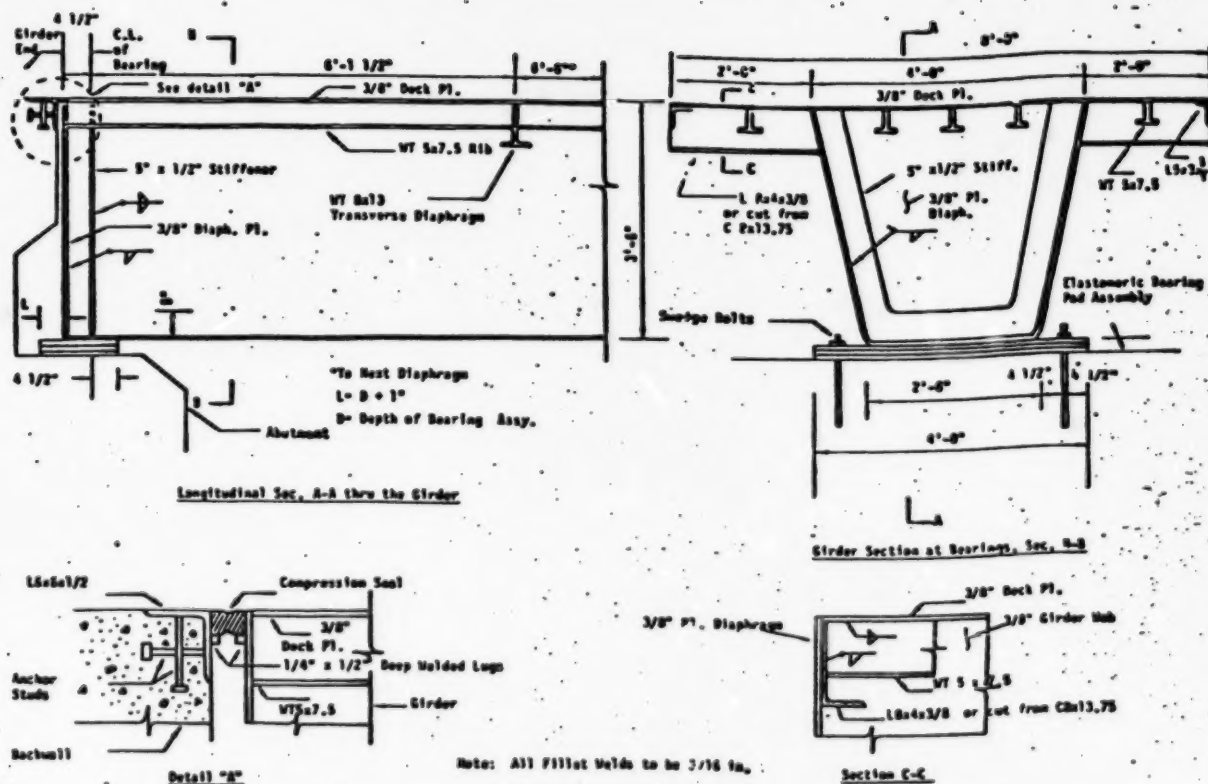


Fig. 4. Bearing end details

construction, there are several other advantages of the steel-deck construction of the proposed system:

1. Due to higher torsional stiffness of a closed section, the lateral distribution of live load is more favourable (smaller distribution factor) in this kind of construction, as compared to that in a conventional slab-stringer system. Consequently, the design bending moment is theoretically smaller for this type of bridge system than for an I-beam bridge system.
2. Since 95% of this type of a bridge system is pre-fabricated under controlled conditions, better quality products can be obtained. This also leads to faster erection of the bridge, which results in avoiding extended detours and traffic tie-ups.
3. Economy in fabrication and erection is achieved by press-forming the trough-section.
4. The weight of a typical unit would enable the use of low capacity equipment for handling, transportation, and erection, resulting in reduced construction cost.
5. Large top flange widths provide safe working space during erection and minimize the number of girders required for a bridge of given width.
6. The shallow box girders are aesthetically appealing, due to their slenderness and clean underside appearance.
7. It is possible to stiffen and strengthen the bridge should it become necessary to raise the load limit.

The girder units as a whole have been designed as a simple beam, whereas the ribs and the supporting diaphragms have been designed according to orthotropic theory. The selection of the 3/8-in. thick top flange is based upon the minimum thickness requirements

from practical considerations. The design of the webs and the bottom flange of the proposed girder has been checked according to the AISI specifications. Typical section properties and stresses for the proposed system are given in Table 1. It can be seen that the design is governed by the live load deflection, rather than strength.

To cut down fabrication costs, a minimum number of welded connections has been suggested. The girder webs do not have any interior or exterior stiffeners. The trough section can be cold-formed on hydraulic press-brakes, thus eliminating the flange-web welding. The ribs and the transverse diaphragms are welded to the deck plate before welding to it the press-formed trough section. All the required welding can be done by an automatic process. Proper welding methods and sequence, minimize the residual stresses and fabrication costs. All welding has been designed for resistance to fatigue.

An extensive literature search revealed that the potential of press-formed box-type bridge systems has not been explored before. To ascertain the practicality of the proposed system, an inquiry with several fabricators/producers, indicate that:

1. Plates up to $1\frac{1}{4}$ -in. thickness and 56-ft. lengths can be brake-formed by hydraulic press brakes currently in use. The widths between the adjacent bends of a section, limited to $38\frac{1}{4}$ -in. by the throat capacity ($36\frac{1}{4}$ -in. horizontal), can be increased to more than 50 in. by minor alterations in the operation.
2. For ASTM A36 steel, the minimum radius that can be obtained for the corners of the trough section is 1.25 times the plate thickness. A smaller radius could develop cracking at the corners.
3. The press-formed section was estimated to be about 35% cheaper than a section welded between the flanges and the webs.

Table 1. Summary of Section Properties and Design Stresses for Press-Formed Steel Box Girder

Parameters	Girder Size					
	6 ft wide			8 ft wide		
	72" x 30"	72" x 36"	72" x 42"	96" x 30"	96" x 36"	96" x 42"
Top flange width, ft	6	6	6	8	8	8
Bottom flange width, ft	2	2	2	2.5	2.5	2.5
Overhang, ft	1.5	1.5	1.5	2	2	2
Total depth, ft	2.5	3.0	3.5	2.5	3.0	3.5
Deck plate thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
Bottom flange thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
Web thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
N.A. from bottom fibers, in.	20.74	24.61	28.38	21.32	24.35	29.27
Moment of inertia, in. ⁴	8,734	13,626	19,775	10,623	16,263	22,806
Sect. modulus, top fibers, S_x , in. ³	943.2	1,196.3	1,451.9	1,223.8	1,396.0	1,791.5
Sect. modulus bottom fibers, S_b , in. ³	421.1	553.7	696.8	498.3	667.9	779.2
Top fiber stress, ksi	7.506	8.045	8.43	6.897	8.189	8.172
Bottom fiber stress, ksi	16.813	17.381	17.566	16.940	17.117	18.789
Max. span capacity, ft	45	55	65	45	55	65
Dead load deflection, in.*	0.14	0.21	0.29	0.15	0.23	0.32
Live load deflection, in.**	0.65	0.825	0.973	0.64	0.82	1.0
Allowable live load deflection, in.†	0.675	0.825	0.975	0.675	0.825	0.975
Weight, plf (no wearing surface)	220	235	250	296	310	330
Weight, psf (no wearing surface)	37	39	42	37	39	41

* Dead load includes 25 psf of wearing surface.

** For maximum span.

† L/800 for maximum span.

Thus, it was concluded that the proposed section is economically feasible from the industry's viewpoint.

Due to the low weight of the girders, only one, or at the most two, 20-ton cranes are required for their installation. During the erection, longitudinal welding between the edges of the top flanges of the adjacent girders along their entire length, to prevent leakage through the longitudinal joint and for transverse load distribution, is the only major on-site work. To overcome unequal deflection of two adjacent girders, temporary fitting aids can be field-welded at the required points which will be used to level the girders. Typical details at the bearings are shown in Fig. 4. Elastomeric bearings pads, are provided at each bearing. Transverse roadway joints between the abutment and the girders are sealed with armored preformed compression seals, as shown in Fig. 4. A typical arrangement with structural tube railing and rail posts is shown in Fig. 1. A rub rail section or a press-formed L-section can be used to provide curbs. Experience with several existing bridges having unprotected checkered or patterned plate steel decks shows that their resistance to corrosion and wear is good. However, their skid-resistance is not satisfactory, particularly when the deck is subject to icing.

Special care has been taken in detailing the system units to avoid sharp corners, stiffeners, bolt heads, etc., in order to minimize the potential for the collection of debris and consequent corrosion, and stress raisers. Accordingly, the corners at the bottom flange of the girders have been rounded off and edges of plates at the girder ends have been eliminated.

Investigations on several bridges with hollow members that have been in service for over 60 years have shown no signs of moisture or corrosion on the inside surfaces. Accordingly, no corrosion-protective treatment is suggested for the interior of the proposed girders. Recent developments in the paint industry claim the service life of some paints to be more than twenty years without any maintenance. Therefore, it is preferable to paint the exterior

faces of the girders, unless weathering steel, is used for fabrication.

As an alternative to the steel plate deck of the proposed system, it is entirely feasible to have a deck made from precast, pre-stressed planks, as shown in Fig. 5. The design of this system is very similar to the all-steel design, except that the deck plate is replaced by 5-in. thick precast pre-stressed concrete panels. A few simple modifications are required in the all-steel design when concrete planks are used:

1. At the junction of the two adjacent beams, shear keys with weld-ties are provided to distribute live load laterally, as shown in Fig. 6.
2. The girders are sealed at the ends. To drain out any moisture that might creep in through the concrete deck, drainage holes are provided in the bottom flange, as shown in Fig. 5. This arrangement is considered adequate to prevent the corrosion of the interior of the girder. The inside faces of steel plates need not be painted.
3. The concrete plank is precast first, with the stud-plate embedded in it. The whole assembly is then shop-welded to the steel trough-section.

Typical bearing details are shown in Fig. 7. The precast concrete planks are nominally prestressed to minimize shrinkage cracks. A summary of section properties and design stresses is given in Table 2 for 6-ft. wide sections. As can be expected, the superstructure weight of this system exceeds the weight of the all-steel system by about 35 psf. This is reflected in the corresponding lower span capacity.

CONCLUSIONS

An all-steel, all-welded box-girder for HS20-44 loading has been presented. For spans up to 65 ft., the proposed superstructure

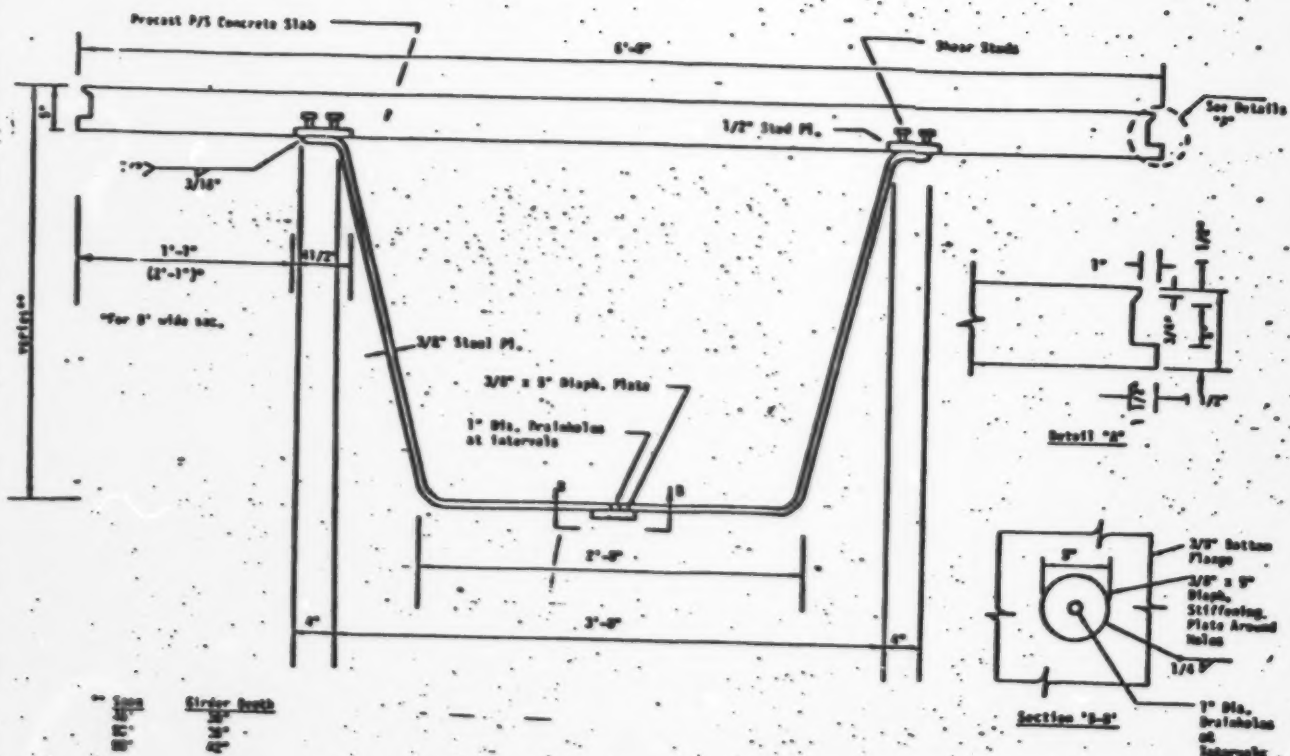


Fig. 5. Composite box girder for 45-ft span

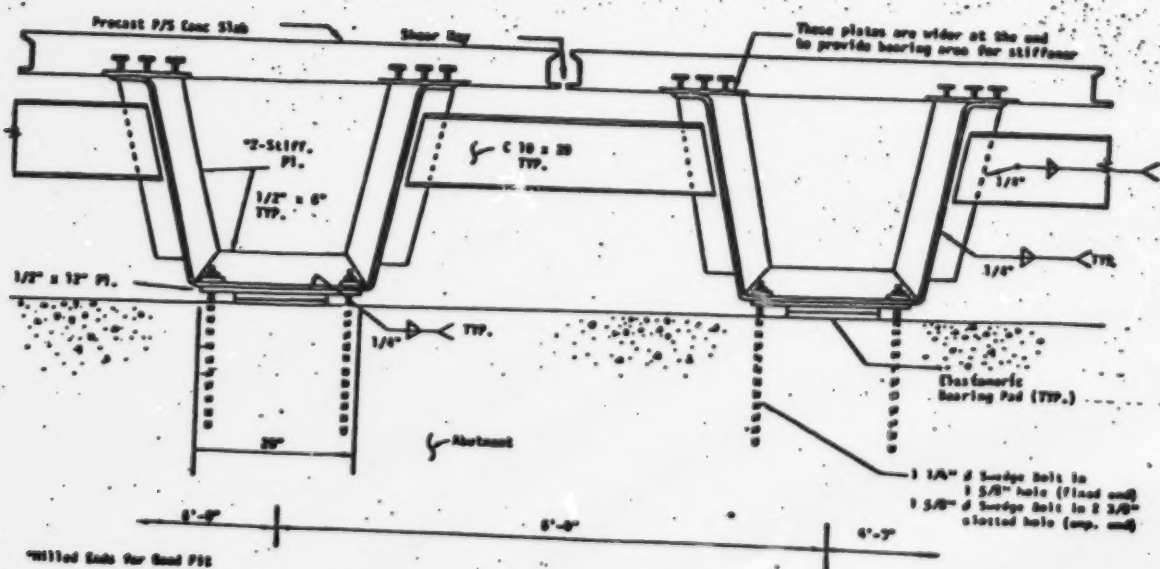
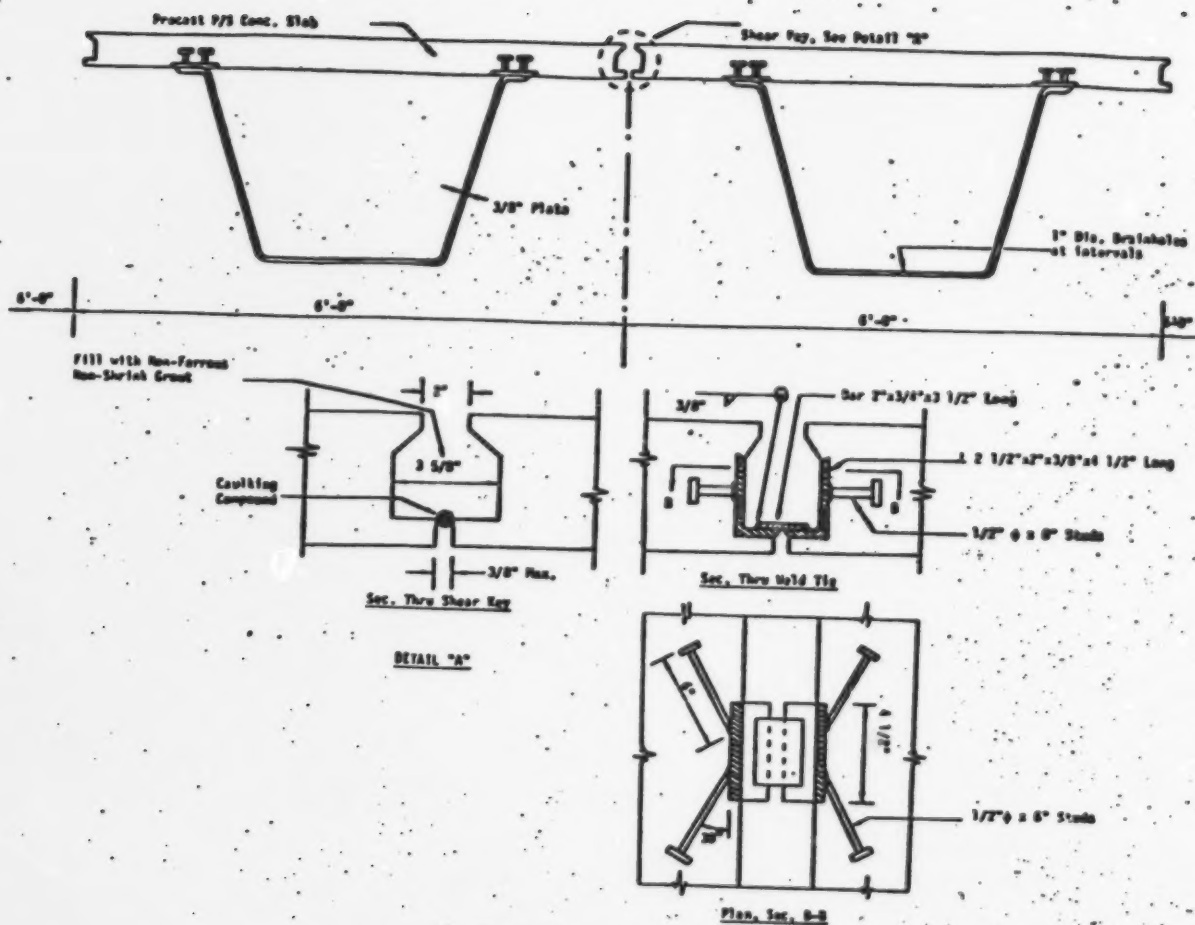


Table 2. Summary of Section Properties and Design Stresses for Composite Box Girder

Parameters	Girder Size		
	6 ft wide		
	72" x 30"	72" x 36"	72" x 42"
Top flange width, ft	6	6	6
Bottom flange width, ft	2	2	2
Overhang, ft	1.5	1.5	1.5
Total depth, in.	30	36	42
Plank thickness, in.	5	5	5
Bottom flange thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
Web thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
N. A. from bottom fibers, in.	19.15	24.46	28.30
Moment of inertia*, in. ⁴	8,304	12,810	18,730
Sect. modulus, top, in. ³	765	1,110	1,375
Sect. modulus, bottom, in. ³	434	523.7	660
Stress, top fibers (conc.), psi	1,197	974	984
Stress, bottom fibers, psi	18,310	18,610	18,510
Maximum span, ft	45	50	58
Deflection (D. L.) in.**	0.44	0.28	0.47
Deflection (L. L.) in.	0.69	0.64	0.71
Allowable deflection, in.†	0.68	0.75	0.87
Weight, plf	500	516	531
Weight, psf	83	86	89

* Composite section, $n = 8$, $f_c' = 4,000$ psi.

** 30 psf superimposed dead load incl. for wearing surface.

† For maximum span, $L/800$.

is light and weighs only about 40-45 psf, as compared with over 100 psf for a conventional slab-stringer bridge system. The system is developed from the view points of segmental assemblage, low initial and maintenance costs, and ease of future expansion.

As an alternative to the steel plate deck, a design with precast, prestressed concrete deck that acts compositely with the steel section is also presented. This system is developed from the viewpoints of segmental assemblage, low initial and maintenance costs, and ease of future expansion.

As an alternative to the steel plate deck, a design with precast, prestressed concrete deck that acts compositely with the steel section is also presented. This system weighs about 85 psf. It is lighter than the conventional slab-stringer system, and lends itself better to modular construction.

Six different girder sections have been presented with a maximum span capacity of 65ft. The girders have widths of 6 and 8 ft. and depths of 30, 38 and 42 in. A suitable combination of these sections can be used to achieve the required bridge width. By increasing the depth of the girders, the system can be used for longer spans.

The proposed system has been designated under the constraints of 6 and 8-ft. wide girder units, from the viewpoint of modular systems. However, further optimization can be achieved by increasing the flange widths to 10 or 12 ft. This would decrease the unit superstructure weight to about 35 psf. With deeper girders and thicker webs and flanges, the system is adaptable to longer spans.

POOR

QUALITY

ORIGINAL

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.2

PRELIMINARY STRUCTURAL DESIGN CRITERIA FOR
HAMILTON ICTS TRANSIT SYSTEM

ABAM LIMITED,
MISSISSAUGA, ONTARIO

MARCH, 1981

NOTES:

1. The Design Criteria and Commentary for the ICTS Guideway Structure is currently being revised. Information included herein is the latest available but is subject to change.
2. The preliminary draft of the Design Criteria relevant to the Steel Alternative Guideway Study issued 25 February 1981 does not, in general, contain paragraph numbers but is page numbered for location of titled sections. Further editing and organization will be evident in subsequent issues.
3. The design examples are composites taken from previous reports provided to UTDC. As such they may be predicated on different design criteria than included herein.
4. This document is to be used exclusively for the development of steel guideway alternative designs.

TABLE OF CONTENTS

	<u>Page</u>
<u>INTRODUCTION</u>	1
<u>OPERATIONAL REQUIREMENTS</u>	2
Introduction	2
Environmental Conditions	2
Vehicle operations	4
Performance Requirements	5
service life	5
durability	5
resistance of accidental occurrences	6
control of vibration effects	6
noise control	6
aesthetics	6
ride quality	7
integration of guideway subsystems	7
drainage	7
<u>STRUCTURAL</u>	8
Forces to be Considered in Design	8
vehicle-related forces	9
Vehicle System Loads	11
Lateral System Loads	12
Longitudinal System Loads	13
Vehicle Mishap Loads	14
Environmental Loads	17
wind load	17
earthquake load	19
general	19
analysis and design	19
restraining features	26
special cases	26
thermal load	26
snow and ice loads	27
other environmental considerations	28
Summary - Loads and Symbols	29
Load Combinations for Design	33

TABLE OF CONTENTS (Continued)

	<u>Page</u>
Strength and Serviceability	35
service load design criteria	36
load factor design criteria	37
stability considerations	39
operational serviceability	39
fatigue criteria	40
<u>TRACKWORK DESIGN</u>	42
Trackwork Structure Interaction Forces	43
Rail Loadings	45
Direct Fixation Fastener Requirements	47
Pull-Apart Gap	52
Fastener Spacing	53
Rail Loadings Applied to Aerial Structure	53
longitudinal restraint forces	53
unbalanced thermal forces	56
<u>LIM DESIGN PARAMETERS</u>	57
General	57
LIM Tolerances	58
<u>POWER RAIL DESIGN PARAMETERS</u>	59
Service Life	59
Thermal Loads	59
<u>TOLERANCES</u>	60
General	60
Beams	62
Walls	62
Columns	62
Trackwork Tolerances	63
rail tolerances	64
Appendix A - References	
Appendix B - Rail Force Calculations	

INTRODUCTION

This document presents design criteria background which has been assembled for the steel alternative guideway from the pertinent information produced in previous ABAM documents. Operational requirements and structural information are necessarily supported by the influence of trackwork design on the structure.

Construction tolerances designated for the ICTS guideway are included with reference made to tolerances actually achieved.

Numbers or paragraphs in  are tentatively included for use in guideway development tasks currently being undertaken. While further analysis or testing will be done to improve upon or verify these values, changes to this document will be made to reflect current values in use at the date of the final report.

OPERATIONAL REQUIREMENTS

INTRODUCTION

The Operational Requirements address the internal and external factors which impact the guideway design. These requirements address environmental conditions, system performance, vehicle operations, speed vs. horizontal curvature and walkways.

Environmental conditions describe wind, rain, snow, ice and temperature conditions. These conditions affect the size of appurtenances and loads on the structure.

System performance information contains a summary of operational characteristics of ICTS. The data is provided to complete the description of external factors influencing the transit system operation. Some operation considerations, such as the possibility of four car trains, impact the guideway design.

Vehicle operations provide the dynamic conditions on the vehicle which result in direct forces on the guideway. Operating conditions result in lateral and longitudinal loading combinations. The speed vs. curve information relates some of these operation conditions to physical layout considerations.

The walkway data interprets the operation requirements and emergency egress requirements of the system. A minimum impact walkway is presented.

ENVIRONMENTAL CONDITIONS

Site independent system environmental factors are given in Table 4.1.1. The values for wind velocity at which vehicles are removed, the operating rainfall intensity, the operating snowfall intensity, and the operating ice accumulation are shown. Site dependent parameters are given in Table 4.1.2.

The values for wind velocity at which vehicles are removed, the operating rainfall intensity, the operating snowfall intensity, and the operating ice accumulation are shown.

<u>ENVIRONMENTAL CONDITION</u>	<u>SI UNITS</u>	<u>ENGLISH UNITS</u>
Wind velocity (steady-state) at which vehicles are removed from the guideway	24 m/s	54 mph
Operating Rainfall Intensity	6.6 cm/hr 12 cm/day	2.6 in/hr 4.8 in/day
Operating Rainfall Intensity	4.4 cm/hr 53 cm/day	1.75 in/hr 21 in/day
Operating Snowfall Intensity on Horizontal and Vertical Surfaces	0.75 cm/hr 1.5 cm/day	0.3 in/hr 0.6 in/day

TABLE 4.1.1 ICTS GUIDEWAY SITE INDEPENDENT DESIGN
ENVIRONMENTAL CONDITIONS

Site dependent environmental factors are given in Table 4.1.2.

	GROUP A	
	<u>SI Units</u>	<u>English Units</u>
Seismic Zone	2	2
Maximum Wind Velocity (Equivalent hourly wind velocity for a 50-year mean recurrence interval)	31 m/s	70 mph
Ambient Temp. Range - Min	-40°C	-40°F
- Max	43°C	110°F
Structure Temp. Range - Rise	30°C	55°F
- Fall	39°C	70°F
Maximum Rainfall Intensity (non-operational)	9.6 cm/hr	3.8 in/hr
Maximum Snowfall (non-operational)	62 cm	24.5 in
Maximum Ice Accretion on Horizontal & Vertical Surfaces (non-operational)	4.0 cm	1.6 in

TABLE 4.1.2 ICTS GUIDEWAY SITE DEPENDENT DESIGN
ENVIRONMENTAL CONDITIONS

Vehicle Operations:

The vehicle operational criteria are given below.

	<u>SI Units</u>	<u>English Units</u>
Maximum Velocity	21.5 m/s	70.54 ft/s
Operating Velocity	20.0 m/s	65.62 ft/s
Maximum Sustained Lateral Acceleration on Passengers	.10 g	
Maximum Sustained Longitudinal Acceleration on Passengers	.13 g	
Maximum Sustained Lateral Jerk on Passengers	.05 g/s	
Maximum Sustained Longitudinal Jerk on Passengers	.10 g/s	
Maximum Superelevation	10%	
Maximum Rate of Change of Superelevation	.25%/m or .035 m/s	.076%/ft .115 ft/s
Maximum Sustained Grade	6% *	
Minimum Sustained Grade	0.5%	
Maximum Sustained Vertical Acceleration on Passengers	0.10 g	
Maximum Sustained Vertical Jerk on Passengers	0.05 g/s	

* Increased grade capability is available but it should be reviewed by vehicle system engineers. For increased grade, vehicle acceleration, velocity and emergency braking and parking may be degraded.

PERFORMANCE REQUIREMENTS

Service Life:

The design life of the guideway structure shall be 50 years. This design life corresponds to 5.475×10^6 passages of a four-car train consist over each lane of the guideway structure. The design histogram of vehicle loads within these cycles is shown below:

Load Description	Vehicle Mass kg (slugs)	No. Cycles	Vehicle Weight kN (kips)	Cumulative No. Cycles
Crush Load and Snow and Ice	22,960 (1570)	3,832	225.0 (50.6)	3,832
Crush Load 108 People	21,160 (1450)	133,043	207.4 (46.6)	136,875
Nominal Load 76 People	18,920 (1295)	136,875	185.4 (41.7)	273,750
Seated Load 30 People	15,700 (1075)	273,750	153.9 (34.6)	547,500
Seats 2/3 Full 20 People	15,000 (1030)	1,916,250	147.0 (33.1)	2,463,750
Seats 1/2 Full 15 People	14,650 (1000)	3,011,250	143.6 (32.3)	5,475,000
		<u>5,475,000</u>		

The fatigue design of the guideway structure is derived from this load histogram. Wearout life is not factored in since the running rails can be replaced.

Durability:

Considerations pertinent to durability are: Load factor assumptions, fatigue, vibration, environmental conditions, material strengths, and quality control and craftsmanship in construction. Thermal effects, snow and ice, salt, rain, wind, seismic effects, and solar radiation are the environmental factors requiring attention.

Guideway design shall also consider the effects of fatigue and wearout.

Resistance to Accidental Occurrences:

Guideway safety features shall be provided with the intention of confining all vehicles under all operating conditions and/or for vehicle suspension failure. Confinement means that all vehicles shall be retained on the guideway in a longitudinal collision between a stationary train and a train travelling at the safe civil speed. Also, vehicles that accidentally derail shall be retained on the guideway.

Columns shall withstand collision by street traffic. Columns shall be able to withstand a force of 1000 kN (225 kips) at one meter (3 ft) above ground level.

Control of Vibration Effects:

It is recognized that harmonic resonance can be induced in an elevated structure when the natural frequencies of the structure and passing vehicles are identical, leading to impairment and possible collapse of the structure. This is normally not a problem with systems supporting vehicles incapable of speeds above 25 m/s (56 mph). Top speed of the ICTS vehicle is 20 m/s $\pm$ 1.5 m/s (45 mph), nonetheless, the guideway span should be designed with a lower limit on natural frequency of 3.0 Hz. This beam frequency is compatible with the requirements for ride comfort for the concrete structure. Refer to the Ontario Highway Bridge Design Code and AASHTO Standard Specifications for Highway Bridges.

Noise Control:

The ICTS structure design shall minimize noise generation. Additionally, noise barriers shall be incorporated where required to reduce to a specified level the transmission of system-generated noise to the surrounding environment. The basic design of the guideway is not compromised by the addition of noise panels.

Aesthetics:

The visual design must consider its impact on the community, both to the potential system user and to residents in the vicinity of the system. Attention shall be given to shape of the elevated guideway, geometrics, surface finishes, and the impact of weathering on surface appearance.

All guideway superstructure elements shall be supported by single shaft columns with the possible exceptions of the switch sections. Wall-type columns shall be avoided if possible.

Any change in guideway superstructure depth and/or any change in guideway construction materials shall be approved by MCL. Change in guideway depth shall be such that visual perception in depth change is a minimum. Change in guideway superstructure depth shall be accomplished by beam taper and there shall be no change in guideway superstructure depth at any pier.

Bottom surface of the guideway superstructure shall be continuous to the greatest extent possible.

Ride Quality:

All guideway elements shall be compatible with the requirement to provide ride quality that exceeds the 2-1/2 hr ISO ride criteria.

The ride comfort criteria are the main factors influencing the geometric design of the guideway. These criteria are expressed as limits on the steady-state acceleration and jerk (rate of change of acceleration). The steady-state lateral and vertical limits are given. They translate into constraints on the degree of curvature and the rate of change of curvature that is permitted in the design of the guideway.

Integration of Guideway Subsystems:

Provisions shall be made for placing and supporting all ICTS subsystems. This includes, but is not limited to, trackwork; power; lighting; vehicle control and communication; warning signs; and fire alarm facilities. Provisions are not made for incorporating existing utility systems within the guideway structure except for lighting or street traffic control as discussed below.

Conduits for power and communication cables for the system, drainage facilities for the guideway, and all other provisions for appurtenances, whether related to the guideway function, understructure lighting, or street traffic control shall be supported within the structure in a manner that facilitates access for maintenance yet removed from public view.

Reinforced concrete slab and parapet dimensions shall be established in consideration of structure strength, conduit installation requirements, provisions for attachment of running rails, LIM rail, power rails, control and communication cable hardware, girder camber variation, and concrete placement procedures.

Drainage:

The guideway shall include adequate provisions for draining moisture from the guideway surfaces. This drainage shall have the capacity to route water to established storm sewers, limiting the quantity falling from the structure to sidewalks and streets below. Joint and interface details shall be designed with waterstops, drip strips and other applicable features to maximize drip protection of the elevated guideway.

STRUCTURAL

FORCES TO BE CONSIDERED IN DESIGN

The forces to be considered in the design of the trackwork and structure fall into three general categories:

- o Those which result from the vehicle and its operations.
- o Those which result directly from environmental effects such as temperature variation.
- o Those which result from the interaction of the support structure and the direct-fixation trackwork.

In order to optimize the design of the support structure and the trackwork as a system, careful attention must be paid to the effects that the reactions from the trackwork have on the support structure and the effects the structural motions of the support structure have on the various elements of the trackwork.

In addition to the magnitude of the applied forces the character of these forces must also be considered in design. In considering the character of the loading we must keep in mind that an appropriate design life for a new urban transportation system is fifty years. A characteristic of intermediate capacity transit is that the trains run at very close headways over many hours of the day every day of the year throughout the life of the system. Thus, we find that a component exposed to the full 50 years of use will experience nearly 95 million passages of a transit vehicle. This represents a significant fatigue loading of the structure and is an important design consideration.

Loadings associated with thermal cycling are applied slowly and much less frequently than those associated with the passage of a vehicle. In 50 years the structures will experience 18,250 diurnal thermal cycles and probably not more than 50 of the full temperature cycles from the maximum design temperature to the minimum design temperature. Loadings which occur during periods of extreme low temperatures must be considered in the context of the possibility for brittle fracture. Loadings associated with maximum design wind will likely only be experienced a few times in the life of the structure. These loadings are relatively rapidly applied and are of short duration.

The loads associated with malfunction impact or crash loadings have a low probability of occurrence for any discrete area of the structure. These loads are impact type loadings of uncertain magnitude.

Vehicle-Related Forces

The ICTS vehicle represents a substantial departure from traditional rail transit vehicles (refer to Figure III-1). The cars are relatively lightweight; the bogies are actively steered to accommodate changes in the track alignment in the smoothest possible manner; propulsion and primary braking is accomplished independently of the wheel to rail interface. All of these things plus the fact that the test track design was completed prior to the time a test vehicle was operated, resulted in the requirement to determine the operational design loads analytically rather than experimentally. In some instances this determination was straightforward; in others the proper design loads to use were subject to considerable uncertainty. Special supplemental attention was given to design criteria which fell into this uncertain category, to assure that the design criteria were appropriately conservative without being unduly conservative.

Vehicle	(kN)		(mm)		(mm)		N (mm)		kN (m)	
	Total Car Wt.-Lbs.	Wheel Load - Lbs.	Wheel Diam.	Load/In. Diam.	Wheel Diam.	Load/In. Diam.	Load/In. Diam.	Load/In. Diam.	Load/In. Ft.	Load/In. Ft.
ICTS	40,360	(179.5)	5,045	(22.44)	18"	(457)	280 lb/in	(49.0)	1.18 K/ft	(17.22)
BART	91,400	(406.6)	11,400	(50.71)	30"	(762)	380 lb/in	(66.5)	1.30 K/ft	(18.97)
MARTA	100,000	(444.8)	12,500	(55.60)	34"	(864)	368 lb/in	(64.3)	1.34 K/ft	(19.56)
WMATA	108,000	(480.4)	13,500	(60.05)	28"	(711)	482 lb/in	(84.4)	1.44 K/ft	(21.02)
U.S. Freight Car	200,000	(889.6)	25,000	(111.20)	36"	(914)	694 lb/in	(121.7)	3.77 K/ft	(55.02)
AREA Cooper E80 (1 locomotive)	568,000	(2526.6)	40,000	(177.93)	--	--	--	--	8.0 K/ft	(116.75)
AASHTO Truck HS 20-44	72,000	(320.3)	16,000*	(71.17)	--	--	--	--	2.5 K/ft	(36.48)
Boeing LRV	103,000	(458.2)	8,580	(38.16)	26"	(660)	330 lb/in	(57.8)	1.45 K/ft	(21.16)

COMPARATIVE LOADINGS OF DIFFERENT RAIL SYSTEMS

FIGURE III-1

*Dual Rubber Truck Tires

Vehicle System Loads:

Dead Load (D):

The dead load consists of all permanently applied loads. These fall into three categories:

- 1) basic beamway,
- 2) secondary structures such as sidewalls, curbs and noise panels, and
- 3) appurtenances such as power rails, conduit, switch and retention features, trackwork, LIM reaction rail, command and control facilities, cables, and emergency walkways.

These loads are itemized

- L - Vertical Live Load. This load includes the weight of the vehicle and its passenger payload. Also of importance are the axle spacings, the number of axles and the expected number of cars in a consist. The minimum is a single car and the maximum is a six-car train consist located on the structure so that the worst stress condition is created.

Crush-loaded vehicles are selected for structure service load and strength design.

- I - Live Load Impact. This load is a function of the vehicle live load, the support structure stiffness characteristics, the vehicle suspension characteristics, the vehicle speed and the character of any support discontinuities which cause random excitation of the vehicle mass.

This is a short duration load but may occur repeatedly in a particular location.

The impact factor I as developed and determined for concrete beams shall be: 10% for CWR line track.

The surface discontinuities in the CWR are minimal due to the tight installation tolerances and the direct fixation of the trackwork to the structure. Surface discontinuities can be built into the track at the switch point and frog structures in turnout areas and at sliding rail expansion joints and broken rail gaps in line track. Accordingly, a higher impact factor shall be used for turnout areas and the structural components at sliding expansion joint areas. The broken rail gap impact occurs randomly and the additional vehicle vertical impact load is adequately covered in the load factors and load combinations under consideration.

The natural frequency response of the elevated structure has not created any significant dynamic amplification with the vehicle. This frequency was computed and was measured.

The additional vertical bending stiffness of the sidewalls and the CWR to the structure is significant. Shorter spans will have a higher natural frequencies and will have less dynamic interaction with the vehicle vertical suspension. The impact factor shall be re-evaluated for special long-span structures and for beams of different cross sections or made from different materials.

No impact factor shall be applied to foundation design loads since the energy is assumed to be dissipated by other structural elements.

Rolling or Lurching Load:

The lateral shifting of vehicle weight from one rail to another is known as rolling or lurching. It is associated primarily with freight train load/low-speed systems with regular rail joints. Since the vehicle has a truck with excellent wheel load equalization characteristics, the vehicle has a low-load capacity and the track does not have regular rail joints, the rolling forces are small. As an upper limit, this lurching load is taken as the maximum load unbalance allowed by the vehicle suspension. One vehicle lurching at one time is considered.

Lateral System Loads:

- 1) Centrifugal Forces (CF & CF2): This force is a function of the vehicle weight, the vehicle speed and the curve radius which produce a fatigue type loading. The vehicle experiences a radial acceleration as it passes through a horizontal curve. The corresponding force imparted to the structure is:

$$CF = \frac{V^2}{R} \frac{L}{g}$$

applied at the vehicle center of gravity.

- 2) Normal Centrifugal Force (CF): Centrifugal forces will be applied simultaneously to all loaded lanes and spans of the guideway structure. The maximum normal centrifugal force is dictated by the ride comfort criteria, and the overspeed control system of the vehicle.
- 3) Extreme Centrifugal Force (CF2): The extreme centrifugal force occurs when the vehicle is under manual control. This force is determined by the velocity that causes an empty vehicle to overturn or 24 m/s whichever is less. This force is applied to only one lane of a dual-lane structure at one time.

Steering and Hunting Force (HF):

In tangent portions of the guideway, the vehicle trucks may oscillate back and forth between rails. This phenomenon is called hunting. The probabilities of two and more wheels in a train contacting the same rail simultaneously are not well documented; three or more seems to be unrealistic. Therefore, the hunting force shall correspond to two wheels contacting the same rail simultaneously. This force is dynamic in nature and may be repeated many times.

Application of the force is at the rail head. This force is generally considered zero in a horizontal curve because of the overriding CF forces.

Hunting forces will be applied to one lane of the dual-lane structure at one time.

Steering Force in Switch:

The steering force through a switch is derived from the maximum lateral acceleration

Longitudinal System Loads (LN, LE):

Longitudinal acceleration or braking loads are specified below:

- 1) LN -- Normal Braking. This force is a function of the vehicle braking rate and the vehicle weight. In areas of the route programmed for braking this is a fatigue type load; in other areas this load is less frequently applied.

Normal Acceleration or Braking: One train at normal deceleration. (LN = 13% of live load.) Live load may or may not include the weight of snow and ice clinging to the vehicle (SIV), depending upon the load combination under consideration.

- 2) LE -- Emergency Braking. This force is a function of the maximum vehicle braking rate and the vehicle weight. The force is rapidly applied and is of short duration. It is infrequently applied on any discrete area of the route.

Emergency Braking: One train at maximum deceleration. The braking curve on each running rail per vehicle is shown

Live load may or may not include the weight of snow and ice clinging to the vehicle (SIV), depending upon the load combination under consideration. Normal braking applies for "Service Load Design," while emergency braking is used for "Load Factor Design,"

Given below are the longitudinal braking load combinations applicable to dual-lane and single-lane design.

Dual-Lane Design:

1) Service Loads:

- o Maximum of six vehicles on one lane braking at the normal service rate.

2) Ultimate Loads:

- o Maximum of six vehicles on one lane braking at the emergency braking rate.
- o Maximum of six vehicles on each lane braking at the emergency rate in opposite directions.

Single-Lane Design:

1) Service Loads:

- o Maximum of six vehicles braking at the normal service rate.

2) Ultimate Loads:

- o Maximum of six vehicles braking at the emergency service rate.

Vehicle Mishap Loads:

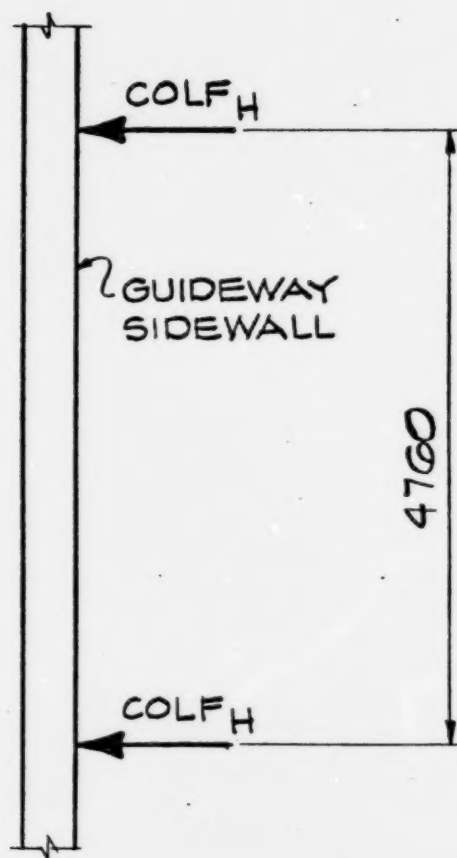
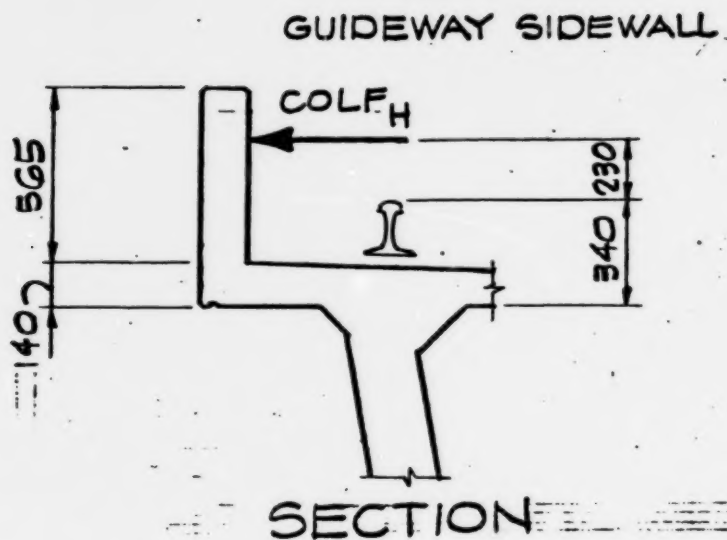
Derailement Loads (DRV):

The derailment loads applied to the structure shall be considered to be less than the collision loads on the structure. For design, no derailment loads are applied to the running rails, the LIM secondary, or power rails.

Vehicle Collision Loads ($COLF_H$ and $COLF_V$):

- 1) Horizontal Collision Loads ($COLF_H$): The vehicle collision loads for design are derived from an analysis of a two-car, four-car or six-car crush-loaded train travelling at a velocity of 5 m/s on a flat horizontal curve with a horizontal radius of 35 m colliding into a stopped two-car empty train or four-car empty train.

The location of these forces is shown on Page 15. The guideway sidewalls shall be designed to resist this energy input and the supporting structure shall be designed to resist the resulting horizontal forces and torsions.



THE MAGNITUDE
TIME OF EACH
FORCE IS GRAPHED
IN FIGURES
5.6.2.1 TO
5.6.2.3

FIGURE 5.6.2.4 DESIGN LOCATION OF ICTS
VEHICLE/GUIDEWAY SIDEWALL HORIZONTAL
FORCE DURING A VEHICLE COLLISION

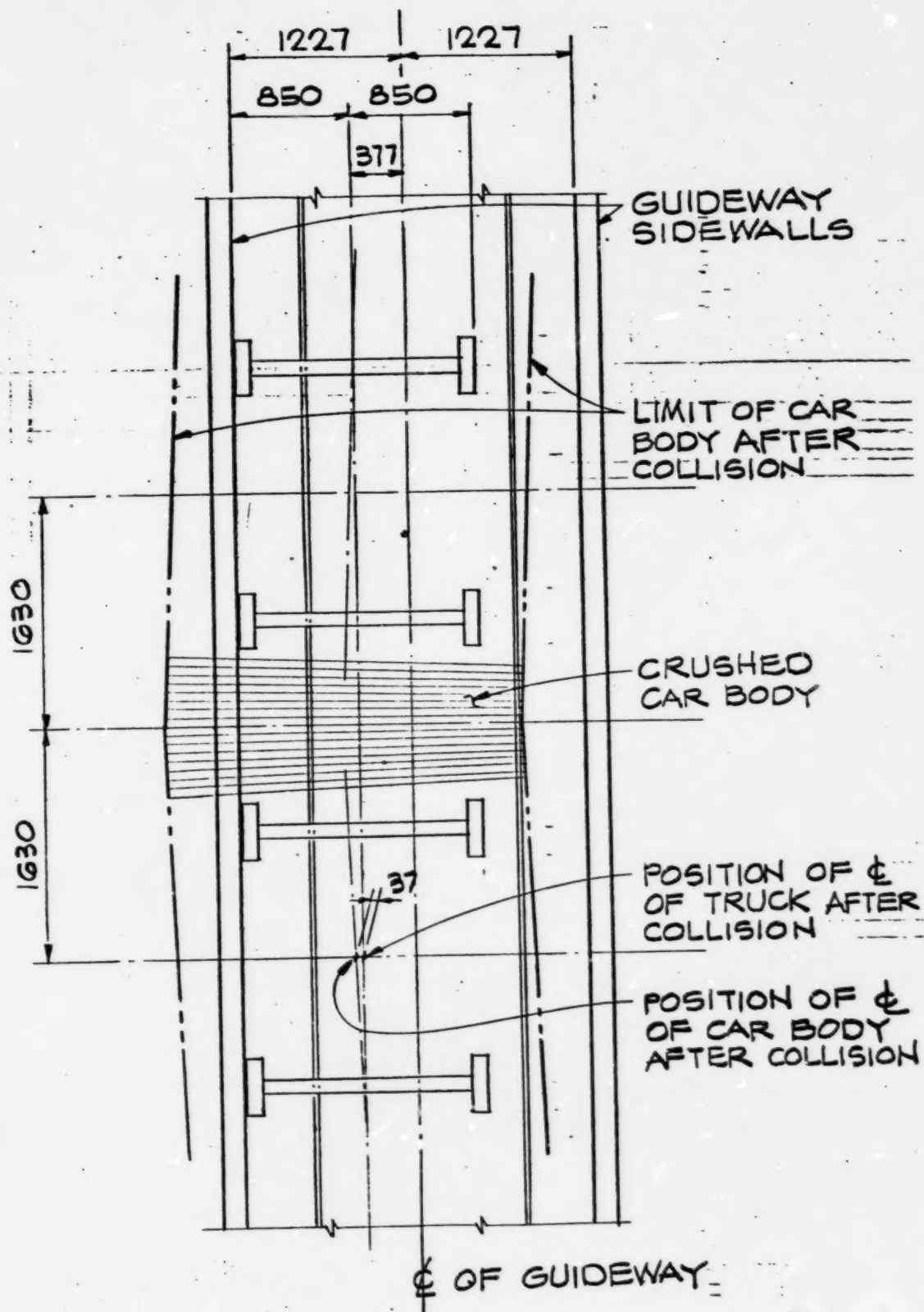


FIGURE 5.G.2.5 DESIGN OFFTRACK POSITION OF ICTS VEHICLES AFTER A VEHICLE COLLISION

- 2) Vertical Collision Loads ($COLF_V$): Additionally, the guideway structure shall be designed to resist the torsion resulting from one end of two cars being off the track centerline as shown on Page 16.
- 3) For design, $COLF_H$ and $COLF_V$ are not applied simultaneously to the guideway structure but are considered to be sequential events.
- 4) For design, no collision loads are applied to the running rails, the LIM secondary, or power rails.

Environmental Loads:

Wind Load (W):

W_1 -- Normal Operation Wind Loading. This loading results from the action of wind on the support structure and on the vehicles. The intensity of the wind is that which can be expected to occur on a relatively frequent basis. These loads are of short duration but are frequent enough that they should be considered in designing for the effects of fatigue.

W_2 -- Extreme Wind on the Structure Only. This wind loading is applied to the structure only. The vehicles are assumed not to be operating during this extreme condition.

For wind load design refer to the 1979 Ontario Highway Bridge Design Code. The ICTS elevated guideway structures shall be designed to resist lateral wind forces and to provide stability against uplift. The lateral wind forces and vertical wind forces specified below shall be applied simultaneously to the guideway structure. The stability criteria is covered. For lateral wind loading, the steady-state wind velocity extremes apply:

- 1) Vehicles Removed from the Guideway: 24 m/s (54 mph) wind acting on superstructure and vehicles.
- 2) Maximum Wind: 31 m/s (70 mph) or 37 m/s (85 mph) wind acting on superstructure only.

The design lateral wind pressure is derived from Reference 2. The design lateral wind pressure is:

$$W_L = K C_e C_g C (1/2 \rho V^2)$$

The design uplift wind pressure is also derived from Reference 2. The design vertical wind pressure is:

$$W_V = C_e C_g C_L (1/2 \rho V^2)$$

This pressure can be applied either upward or downward.

where

V = steady-state wind velocity in m/s (ft/s)

The maximum steady-state design wind velocity is the equivalent hourly wind velocity for a 50-year mean recurrence interval 30-ft above ground level. Wind velocity data for U.S. cities is taken from Reference 34 adjusted by a time duration factor on velocity of 1.3 between the fastest mile of wind and the equivalent hourly wind velocity.

Data for Canadian cities is found from Reference 2.

p = mass density of air at a standard temperature and pressure.

$$= \frac{1.230 \text{ sec}^2}{m^4} - Nt (.00238 \text{ slugs/ft}^3)$$

The approximate equation for the mass density of air is:

$$p = \frac{0.0411 P_b}{T_A}$$

p = mass density of air in slugs/ft³

P_b = barometric pressure in inches of mercury

T_A = Temperature in deg. F absolute
 $T_F + 460^\circ\text{F}$

The density of air used in the criteria is for $P_b = 30$ in mercury and $T_F = 60^\circ\text{F}$. Higher air densities occur at very cold temperatures.

K = factor for type of guideway track

= 1.2 for dual-lane structures

= 1.0 for single-lane structures

This factor accounts for a probable greater exposure area to a wind that is not horizontal with the ground for a dual-lane structure.

C_e = exposure factor

The exposure factor is found from the following table reported in Reference 2.

<u>Height H, m (ft)</u>	<u>Exposure Coefficient C_e</u>
0 to 10 (0 to 33)	1.0
Over 10 to 16 (32 to 52)	1.1
Over 16 to 25 (52 to 82)	1.2
Over 25 to 37 (82 to 121)	1.3
Over 37 to 54 (121 to 177)	1.4
Over 54 to 76 (177 to 249)	1.5
Over 76 to 105 (249 to 344)	1.6

$$C_g = \text{gust factor} = 2.0$$

The gust factor for a structure is related to the time it takes to engulf a structure and to establish flow lines around the structure. The significant gust duration for a structure is approximately that time it takes the wind to travel twice the structure dimension in the direction of the wind. Gusts of 1-second duration at the design wind speeds are sufficient to entirely engulf the guideway structure. A gust factor of 2.0 is used for design. This factor is in accordance with Reference 2.

$$C_p = \text{external pressure coefficient} = 1.5 \text{ or } 2.0$$

This accounts for direct pressure and lee-side suction. The range of values is from .75 to 3.6. Based on Figures C1-17 and C1-19 of Reference 27, a value of 2.0 seems most reasonable for the structure only and 1.5 for the structure plus vehicles if the vehicles have rounded corners. See Reference 25 for supplemental information.

$$C_L = \text{lift coefficient} = 1.0$$

This coefficient was reported in Reference 18 and Reference 2 was based on wind tests of girder bridges.

Tabulated values for wind pressure are given on page 32.

Earthquake Load (EQ):

General: Earthquake forces are generated in guideway structures in response to an earthquake ground motion anticipated for a given location. This criteria has been developed from References 1, 14, and 32.

Analysis and Design: EQ for guideway structures shall be determined directly as individual member forces by dynamic analysis. This method utilizes a model analysis based on the application of a response spectrum of ground acceleration to a lumped mass space frame model of the structure. The space frame model shall adequately account for intermediate hinges and longitudinal and lateral restrainers. The model shall also account for soil spring constants required to develop column base reactions. Five percent (5%)

damped elastic response curves shown on the following pages are used as the response spectrum loading. These curves are dependent on the depth of alluvium over bedrock and the ground acceleration. The 5% damping factor has been found to adequately account for energy dissipation due to friction between adjacent components, movement of foundations through soil soil, and internal molecular friction during deformations.

The design ground acceleration shall be :

0.22 g for Zone 2 - Group A

The ARS response spectra for various soil depths are shown on Figures 5.7.2.1, 5.7.2.2, 5.7.2.3, and 5.7.2.4. These spectra are taken from Reference 14.

Design forces in individual elements shall then be determined by dividing the resulting elastic forces by an appropriate factor, Z , from Figure 5.7.2.5.

In no case shall the design force on a column or pier be less than the following fraction of the DL axial force on a member.

.06 for structures in Zone 2

Although guideways may potentially be designed to resist earthquakes elastically, it is generally not economically feasible. It is also inconsistent with the current design philosophy, which relies on energy absorption to avoid total collapse of a guideway in a major earthquake. This energy absorption in the post-elastic range is commonly handled through the use of a so-called ductility factor, or response modification factor. Ideally, bridge structures should be designed so that the earthquake energy will be dissipated by the individual members acting in a ductile manner, avoiding brittle shear failures. This is, however, not possible in all cases for guideway design, since some of the components may behave in a non-ductile fashion. Since the ductility levels may vary for the individual components of a bridge, reduction of the elastic response spectrums for design may be somewhat misleading and may result in some members being underdesigned. Using elastic design response spectrums to predict the overall structure response and then designing the ductile components to absorb the required energy appears, at the present time, to be the most rational approach that can be used using elastic dynamic analysis capabilities. This approach also conveys how a guideway would actually behave during an earthquake.

Well confined ductile single columns are common features in guideway structures.

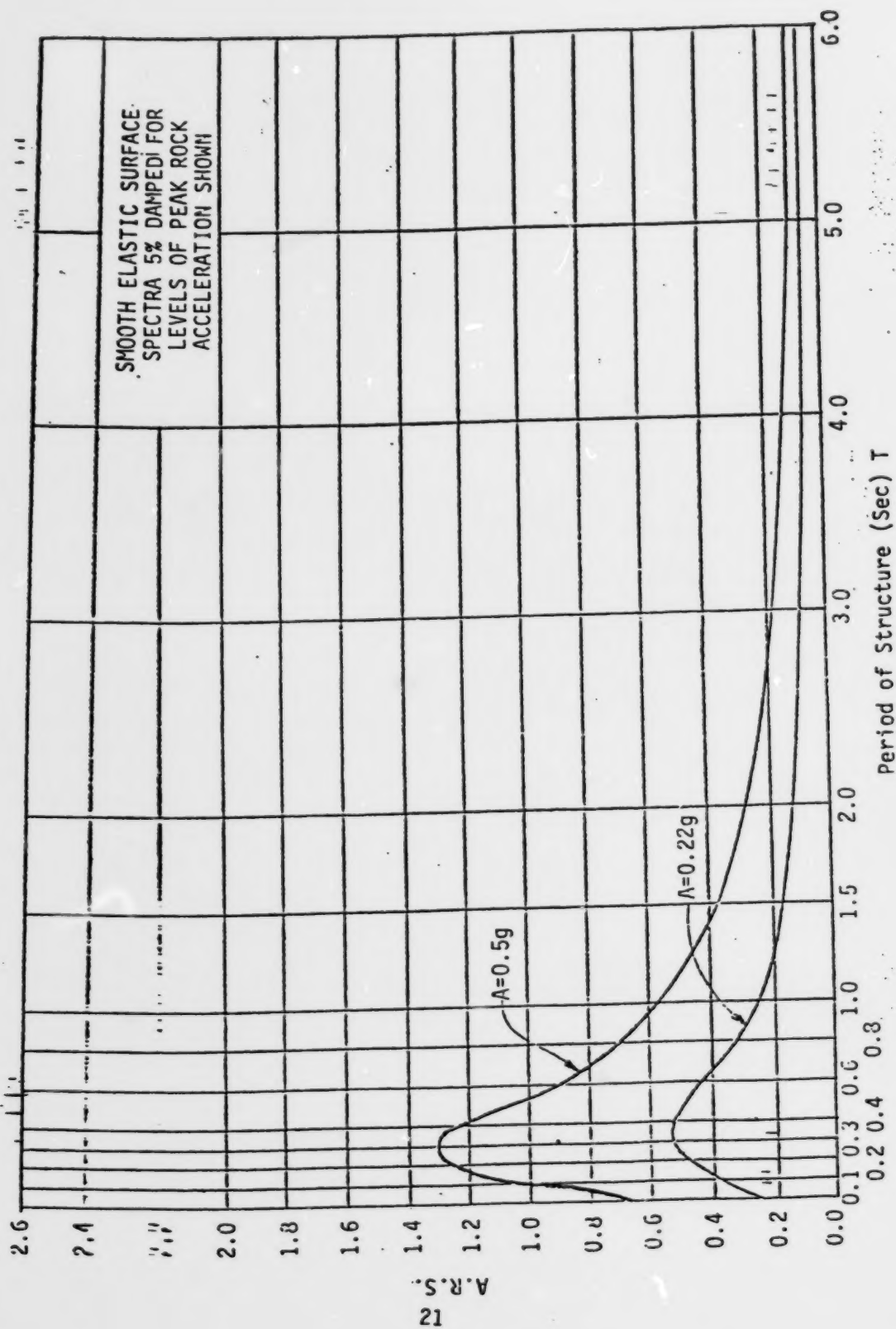


FIGURE 5.7.2.1 0-10' ALLUVIUM

A.R.S. SPECTRA

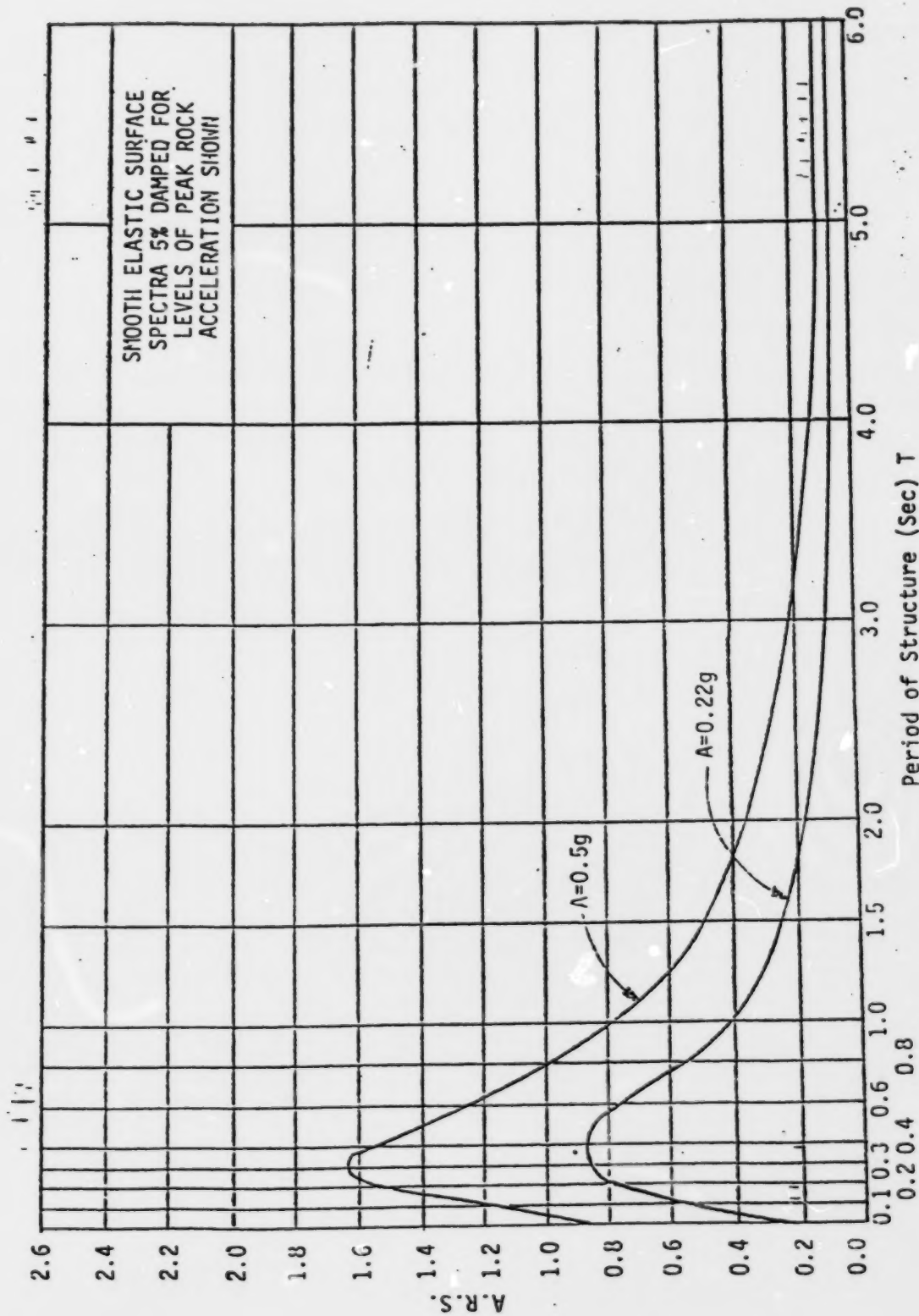


FIGURE 5.7.2.2 10'-30' ALLUVIUM

A.R.S. SPECTRA

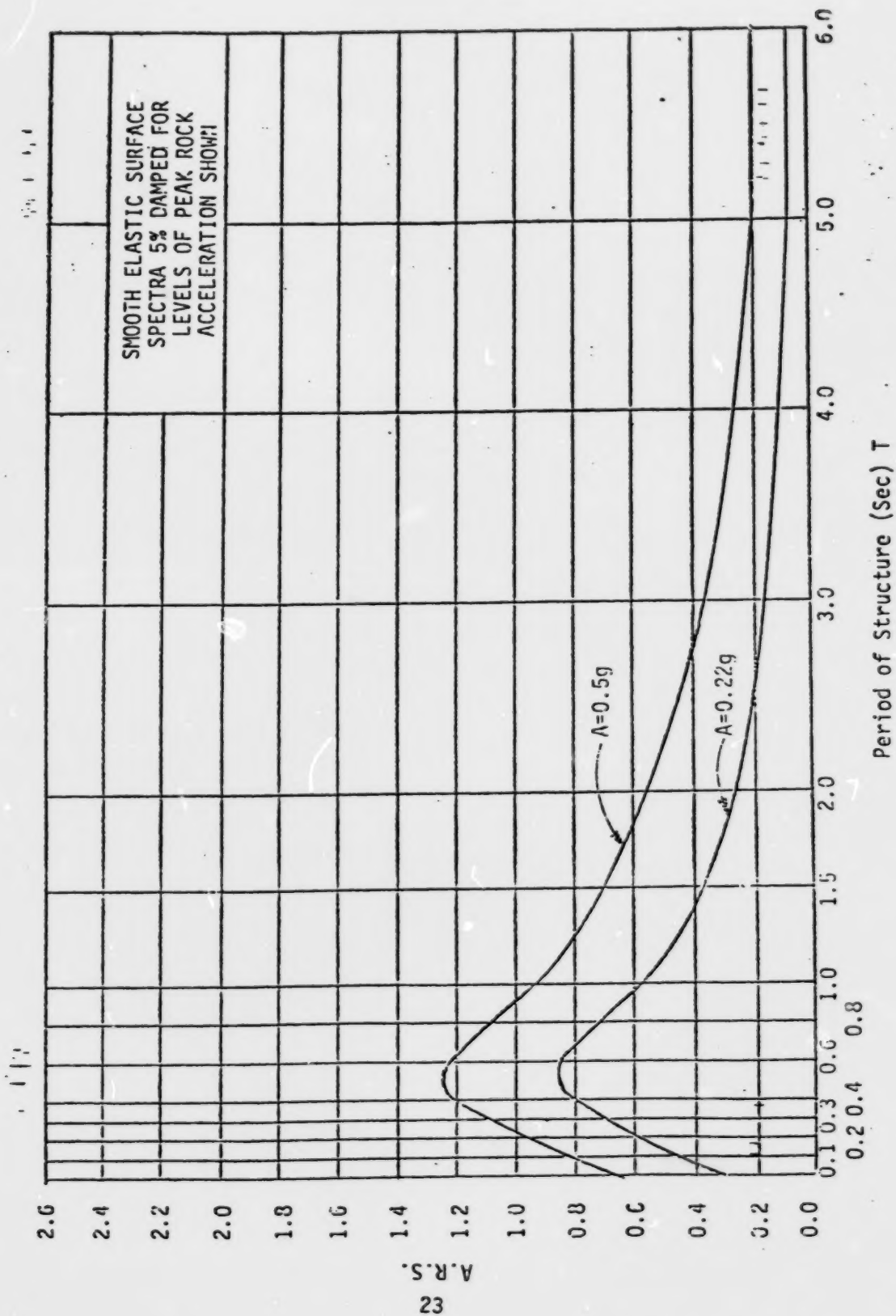


FIGURE 5.7.2.3 90'-150' ALLUVIUM

A.R.S. SPECTRA

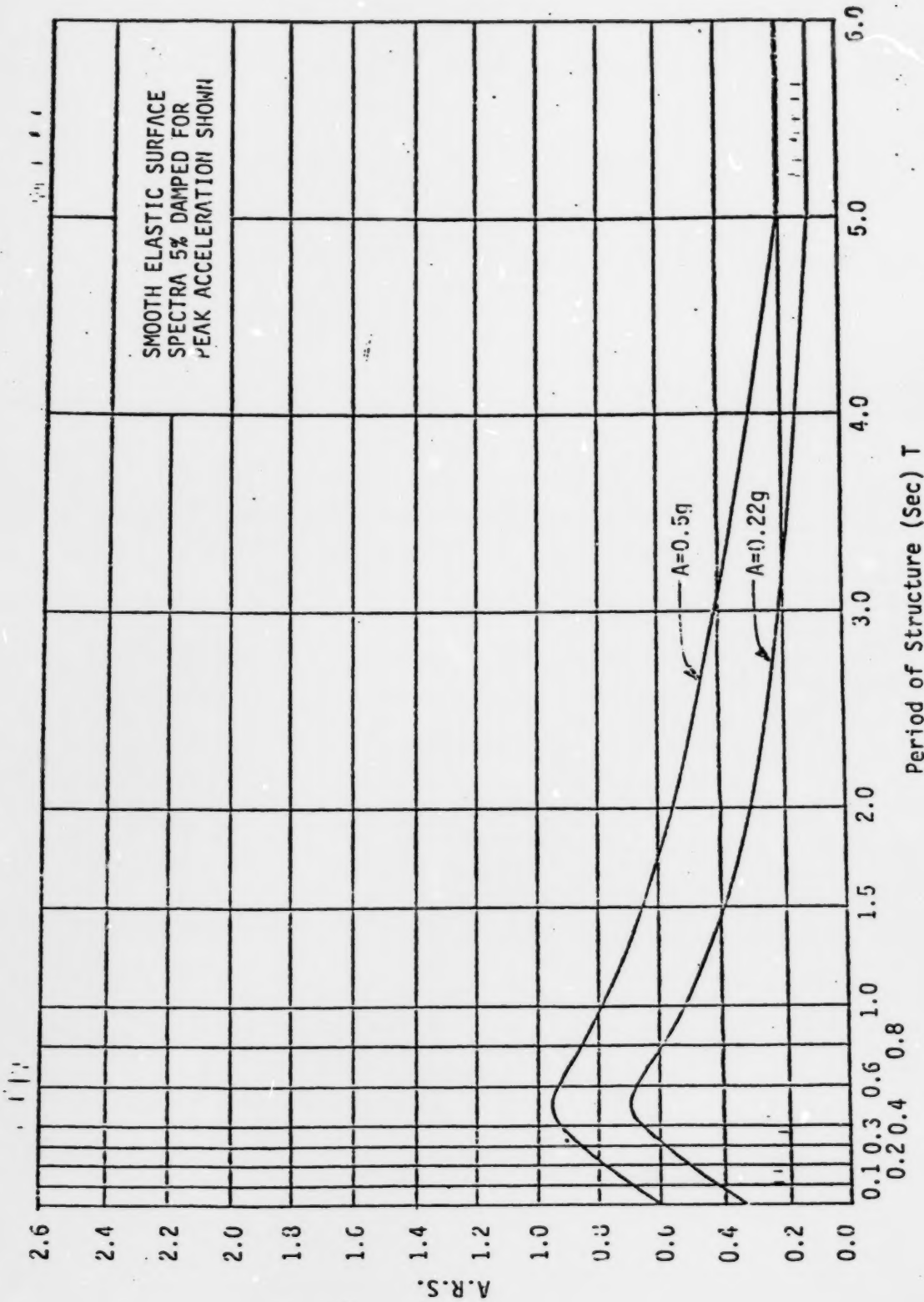


FIGURE 5.7.2.4 150' ALLUVIUM
A.R.S. SPECTRA

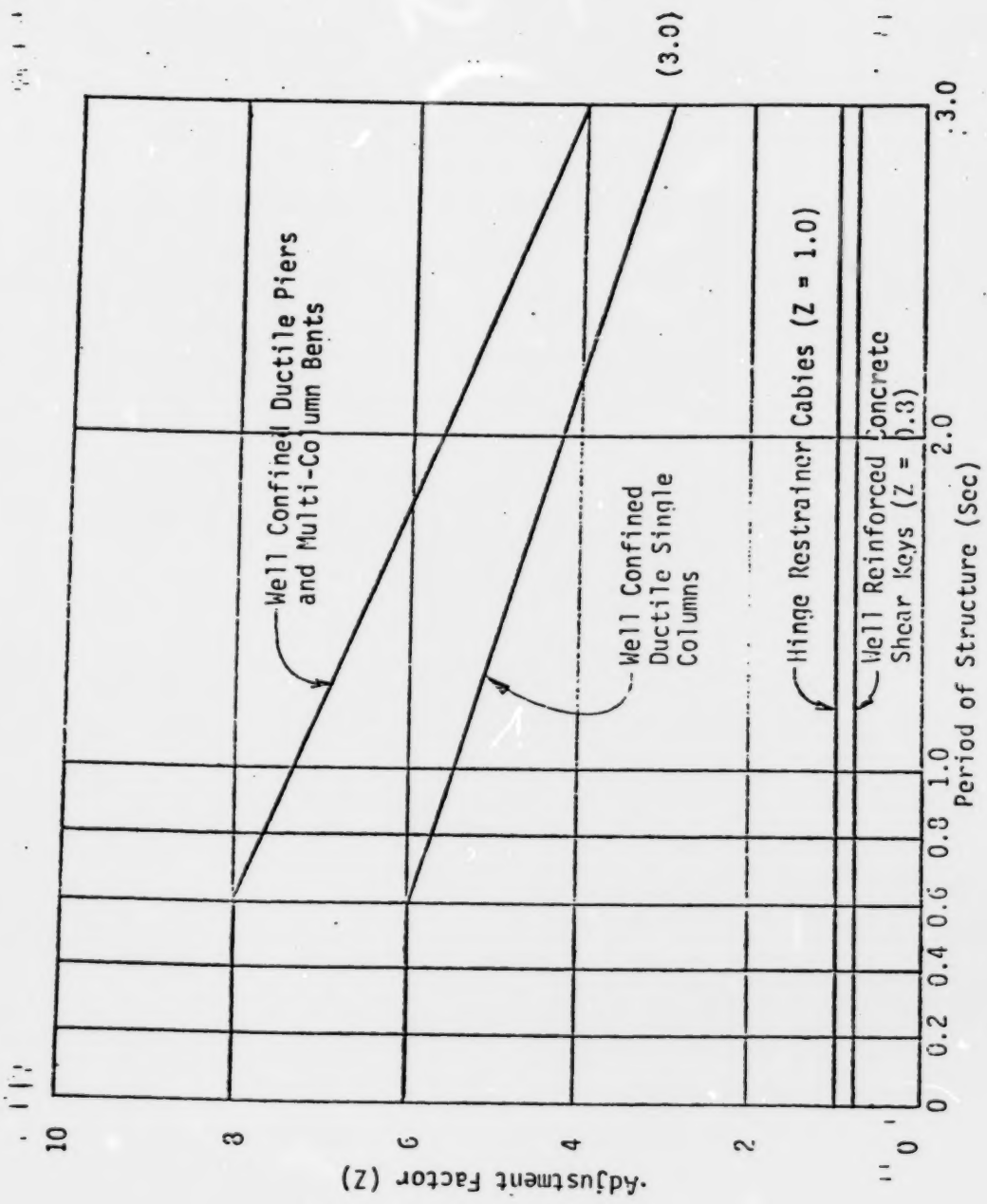


FIGURE 5.7.2.5 ADJUSTMENT FOR DUCTILITY AND RISK ASSESSMENT

These columns shall be designed:

- 1) So that there is adequate plastic-rotation capacity before the strain hardening region in the reinforcing steel is reached.
- 2) So that the column does not fail in a brittle shear mode before the plastic-moment capacity of the column is developed.

Restraining Features: Hinge restrainers, shear keys and other restraining features to limit structure displacement, preferably shall be designed for EQ determined by the response spectrum method applied to the structure as a whole, but in no case for less than:

$$EQ = 0.25 \text{ (Contributing DL)}$$

Contributing DL is that portion of total structure DL whose response to earthquake motions can directly effect the feature under consideration. For guideway structures, contributing DL may also include a contribution from loads on restrainers beyond the immediately adjacent frames or spans.

When the response spectrum method is used, Z shall be taken from Figure 5.7.2.5 using the curve for the feature under consideration.

The use of the continuously welded rails to limit structure displacement shall also be considered.

Special Cases: Sites adjacent to active faults, sites with unusual geologic conditions, branching switch structures, and other unusual structures will be considered special cases. In such cases, structures shall be evaluated for EQ by current seismicity, soil response, structure response, and dynamic analysis techniques.

T -- Thermal Forces. These forces result from the thermal expansion and contraction of the various elements of the system. They may be direct stresses and forces caused by thermal changes or they may be the result of reacting thermal forces in other members. Lower magnitude thermal forces result from diurnal temperature variations. Changes in these forces are developed over a period of hours. Maximum variations in thermal force occur on a seasonal basis and the trend can be considered as a sustained load for purposes of design.

Provision shall be made for stresses or movements in the structure resulting from variations in ambient temperature. Thermal activity affects the structure in two manners:

- 1) Seasonal: Beamway elongation or shortening, and column displacement, due to uniform temperature changes. The coefficient of thermal expansion for concrete structures shall be $9.9 \times 10^{-6}/^{\circ}\text{C}$ ($5.5 \times 10^{-6}/^{\circ}\text{F}$). The ambient temperature ranges for environmental-Group A are shown in Table 4.1.2. Considering the lag between air temperature and the interior temperature of the concrete beamway, the structure shall be designed for the temperature rise and temperature fall shown in Table 4.1.2.

- 2) Local: Beamway cambering or sagging, and column rotation, due to temperature variations within the beamway. The design thermal gradient shall be 22°C (40°F) through the deck flange. The temperature throughout the remainder of the beam section is considered uniform and equal to the temperature at the bottom surface of the deck flange.

- Snow and Ice Loads (SIV, SIG):

- SIV/SIG -- Snow and Ice Loadings. This loading is an effective addition to the vehicle live load and to the support structure dead load to account for the additional weight of snow and ice carried by the system during the winter months.

SIV = Snow and ice load surcharge to vehicle live loading.

SIG = Snow and ice load surcharge to guideway dead loading.

Snow and ice accumulation shall be in accordance with the values given in Tables 4.1.1 and 4.1.2.

Snow and ice loading falls into one of three categories:

- 1) Ice Clinging to the Revenue Vehicle (SIV)
- 2) Normal Operation: Maximum snow and ice accumulation on the guideway (SIG).

References 11 and 12 were used as a guideline for determining the equivalent load. This load is calculated from the following assumptions and conclusions:

- o Ice accretion on deck slab to the top of the running rail.
- o No ice on vertical surfaces.
- o Relative density of ice = .975.
- o Snow accumulation to an average depth above ice of 15 cm (5.9 in).
- o Relative density of snow = .20.

Load, then, is a function of rail size. The snow/ice load based on a 115# AREA rail.

- 3) Suspended Operation: The snow/ice load is less than that caused by normal operating vehicle live loads; thus, it is not included in the criteria for "Strength and Serviceability."

Other Environmental Considerations:

Other loads listed below may apply to the ICTS structure. These loads shall be computed by a rational method and applied to the structure in a manner consistent with their occurrence. They are:

SF - streamflow

EP - earth pressure

B - buoyancy

SE - settlement

In addition to environmental loads, environmental effects of a non-loading nature shall be dealt with in a rational manner:

- 1) Rain precipitation for normal operations and extreme rainfall intensities as given in Tables 4.1.1 and 4.1.2. The drainage system shall be designed in accordance with the above precipitation rates.
- 2) Consideration shall be given to the service life of the structure as affected by electromagnetic fields.
- 3) The design life of the structure shall apply to salt atmospheres as found in coastal areas or in areas where salt is used to de-ice nearby highways. It is not anticipated that salt will be used for snow and ice control.

Summary - Loads and Symbols:

Below is summary of loads and their values to be applied to "Service Load Design," and "Load Factor Design."

D = Dead Load

	Concrete Trapezoidal Box Cross Section		Steel	
	<u>kN/m</u>	<u>kip/ft</u>	<u>kN/m</u>	<u>kip/ft</u>
Basic Beamway	29.58	2.038	To be determined	
Side Walls	0		0	
Noise Walls	Not included in Basic Design			
Power Rails	0.36	0.025	0.36	0.025
Conduits and Cables	0.29	0.020	0.29	0.020
LIM Secondary and Attachments	1.09	.075	1.09	.075
ATC Loop	0		0	
Running Rails and Attachments (115# REA)	1.60	0.110	1.60	0.110
Walkways	0.29	0.020	0.29	.020
TOTAL	32.21	2.288	36.68	2.515

L = Vertical live load exerted by a train consist of one to six loaded vehicles on one or both lanes of the dual guideway. Each crush vehicle load is 21,160 kg, not including snow and ice (SIV).

I = Vertical impact associated with live load under consideration which may or may not include snow and ice (SW). Developed at STF and determined for concrete beams.

I = 0.10 L + .10 SIV or 0.10 L

In switch areas and for structural components over sliding expansion joints in the rail.

I = 0.30 L + 0.30 SIV or 0.30 L

Lurching Load: The design lurching load applied to the structure is the torque associated with an unbalance in wheel load on vehicle of 84% to 116% of the normal wheel load.

Centrifugal Forces (CF, CF2):

CF = Normal Centrifugal Force

$$= \left(\frac{V^2}{Rg} - e \right) L \text{ on beams}$$

$$= \left(\frac{V^2}{Rg} \right) L \text{ on columns}$$

applied at the vehicle center of gravity, where

V = Vehicle velocity; the design velocity shall be the design velocity permitted on the guideway structure to keep the lateral accelerations within the ride comfort criteria

plus the incremental velocity allowed by the vehicle speed control system.

R = Radius

g = Acceleration due to gravity

L = Vehicle live load (may or may not include snow and ice)

e = Superelevation

TABLE OF NORMAL CENTRIFUGAL FORCE

<u>R</u> <u>m</u> <u>(ft)</u>	<u>e</u>	<u>V_{des}</u> <u>m/s</u> <u>(ft/s)</u>	<u>V = V_{des}</u> <u>+ V_{over speed}</u> <u>m/s (ft/s)</u>	<u>Horizontal Acceleration</u> <u>-g-</u>
150 (492)	0.10	17.15 (56.3)	18.65 (61.2)	.237
35 (115)	0	5.86 (19.2)	6.86 (22.5)	.137

<u>V_{des}</u>	<u>V_{over speed}</u>
0 - 3.8 m/s	0.50 m/s
3.8 - 14.0 m/s	1.00 m/s
14.0 - 20.0 m/s	1.50 m/s

CF2 = Extreme Centrifugal Force:

R m (ft)	e	V_{max} m/s (ft/s)	Equivalent Horizontal Acceleration on Crush-Loaded Vehicle $-g-$
150 (492)	0.10	24.0 (78.7)	0.252
35 (115)	0	16.9 (55.4)	0.534

V_{max} is computed on a velocity which causes tipping of an empty vehicle or a 24 m/s on an empty vehicle whichever is less.

HF = Steering and Hunting Forces:

$$\frac{HF}{2/8 (L + SIV)} = 1.0$$

$$HF = 51.9 \text{ kN } (11.7^K) \text{ for two axles of one vehicle } (SIV = 0)$$

$$HF = 56.3 \text{ kN } (12.7^K) \text{ for two axles of one vehicle } (SIV = \text{as given below}).$$

LN = Normal Longitudinal Braking or Acceleration Force:

$$LN = 0.13 (L) = 27.0 \text{ kN } (6.1^K) \text{ per vehicle, } SIV = 0.$$

$$LN = 0.13 (L + SIV) = 29.3 \text{ kN } (6.6^K) \text{ per vehicle, } SIV = \text{as given below.}$$

LE = Emergency Longitudinal Braking Force:

$$LE = \text{A maximum of } 46.0 \text{ kN } (10.34^K) \text{ per rail per vehicle.}$$

COLH_H
and
COLF_V

= Horizontal and Vertical Vehicle Collision Loads:

$$COLH_H = 2 - 74.3 \text{ kN } (16.7^K) \text{ concentrated horizontal loads.}$$

$$COLF_V = 86.0 \text{ kN-m } (63.4 \text{ k-ft}) \text{ concentrated torque.}$$

The plastic moment strength of the sidewall limits the horizontal force applied to the guideway structures to 16.7^K during a vehicle collision. These forces are predicted on using a sidewall 150 mm (6 in) thick with vertical reinforcing of No. 5 bars at 150 mm (6 in) on center placed in the middle of wall section.

W_1 = Wind pressure at which vehicles are removed from the guideway. Mean wind velocity = 24 m/s (54 mph).

W_2 = Maximum wind pressure on structure only. Mean wind velocity = 31 m/s (70 mph) or 37 m/s (85 mph).

Condition	Vehicle	Structure	
		Lateral	Vertical
W_1 Single Lane	1.16 kpa 24.3 psf	1.16 kpa 32.7 psf	0.78 kpa 16.4 psf
W_1 Dual Lane	1.39 kpa 29.2 psf	1.87 kpa 39.2 psf	0.78 kpa 19.6 psf
W_2 Single Lane Group A		2.62 kpa 55.0 psf	1.31 kpa 27.5 psf
W_2 Dual Lane Group A		3.14 kpa 66.0 psf	1.57 kpa 33.0 psf

All values are based on $C_e = 1.1$

EQ = Earthquake Loads

T = Thermal Effect

SIV = Ice Load Attached to the Vehicle (Normal Operations)
= 17.66 kN (4.0 k) per vehicle

SIG = Snow and Ice Load on the Guideway (Normal Operations)

SIG = 1150 Pa (24.1 psf) for 115 lb REA rail

R = Effect of Rib-shortening due to prestressing

S = Effect of Concrete Shrinkage

ST = Accidental Load due to collision of street traffic =
1000 kN (2.25 k) load applied to columns at a height
of 1 m (3 ft) above ground level.

LR_1, LR_2, RT_1, RT_2 -- Radial and Longitudinal Restraint Forces. These forces result primarily from the interaction of the aerial support structure thermal motion and the stationary continuous welded rail. The forces result from the longitudinal frictional restraint characteristics of the direct-fixation fasteners. LR is associated with tangent structures and RT is associated with curved structures.

RT_1, LR_1 = Longitudinal Restraint Force (Normal Operations) LR_1 = TBD

RT_2, LR_2 = Longitudinal Restraint Force (Malfunction Condition) LR_2 = TBD

The above forces are a function of radius of curve, rail size, fastener longitudinal restraint, and temperature drop or rise with respect to rail design stress free temperature.

BR = Broken Rail Force. The force transmitted to the support structure when a rail pull-apart occurs. The forces are derived from considerations of interaction of the structure and trackwork.

RT_1, RT_2 = Rail/Structure Interaction Forces in Curved Structures. These forces are a function of radius of curve, central angle of curve, rail size, rail fastener restraint, and temperature drop with respect to rail setting temperature. RT_1 is computed using the normal operations fastener restraint. RT_2 is computed using the malfunction fastener restraint. For a further description of these rail forces, see Appendix B.

The structural support concepts employed to support the trackwork should be developed with these interaction effects in mind. Concepts should be developed which minimize the adverse effects of these interactions on the system as a whole. The design for the effects of interaction between the trackwork and the support structure is discussed in Appendix B.

LOAD COMBINATIONS FOR DESIGN

Once the different forces acting on the trackwork and its support structure have been identified and their magnitude and character assessed the problem then becomes to determine how these various loadings should be combined for use in the design of the structure.

Design loading conditions have been developed by the various agencies who prepare codes for structures. The Ontario Ministry of Transportation and Communications has developed design loadings for use in the design of highway bridge structures. The American Association of State Highway and Transportation Officials (AASHTO) have developed design loading conditions for use in the design of highway bridge structures. The American Railway Engineering Association (AREA) have developed design loading conditions for use in the design of railway bridge structures for heavy rail service. The American Concrete Institute (ACI) has developed design loading conditions of a general nature for use in design of concrete structures.

None of the existing codes were found to be directly applicable to the design of a transit system with the characteristics of ICTS. For this reason, a program was instituted to modify the code design loading conditions so that the unique aspects of design for ICTS would be reflected in the design load combinations. For purposes of design, special attention was given to the manner in which the trackwork/structure interaction forces were combined with the other operational and environmental forces.

The design of the ICTS support guideway beams is composed of two basic steps. The first step is to design the beam for maximum serviceability at normal operating conditions. For prestressed concrete this implies a working stress design with particular attention given to deflection and camber under normal, not maximum, loads.

During the design life of the system, the structure will be subjected to abnormally high forces as result of extreme temperature variations, earthquakes, storm winds, and continuous vehicle loads due to possible system malfunctions. These forces, singly and in combination, obviously should not cause structural failure, but neither should they dictate the design comfort criteria. Thus, the second step in the design process is to perform a strength design based on an ultimate strength analysis.

The performance of the ultimate strength analysis gives direct insight into the actual safety factors that exist against failure of the structure. The working stress design on the other hand focuses on things such as the detail structural deflections, and design crack widths which respectively have an important effect on the ride comfort considerations of the support structure and on its long-term durability.

Columns, crossheads, foundations and connections are designed by the load factor design criteria to provide an ultimate strength capacity sufficient to carry the factored load combinations.

Strength and Serviceability

For strength and serviceability requirements refer to the Ontario Building Code. Criteria presented below are specific to the concrete structure but may serve as guidelines for steel design. The prestressed concrete beam elements shall be designed using the Service Load Design (SLD) criteria

SLD employs load combinations representing the normal crush-loaded operation. Beams thus designed shall then be checked for conformance with ultimate load conditions utilizing the Load Factor Design (LFD) criteria.

Columns, crossheads, foundations and connections shall be designed by the LFD criteria to provide an ultimate strength capacity sufficient to carry the factored load combinations.

Particular care shall be taken to ensure that the beam and cross-head elements of the dual guideway are provided with adequate reserve strength to carry the torsional stresses. Design for torsion shall be in accordance with the ACI Committee 443 report (Reference 6).

Service Load Design Criteria: (Prestressed Concrete Section 5)

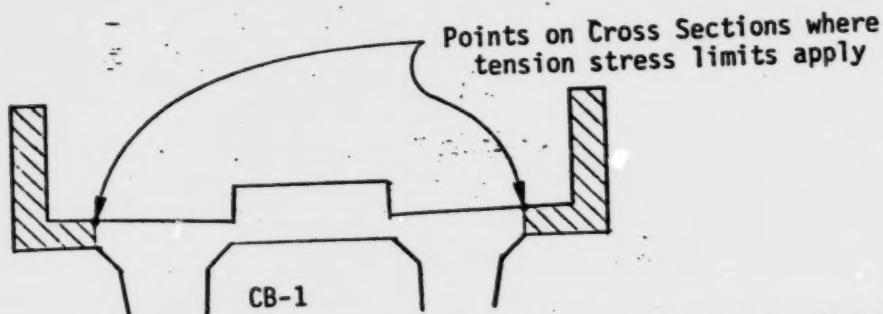
A $3\sqrt{f'_c}$ (English units) tension criteria for monolithic concrete and a zero tension criteria across construction joints is employed for purposes of emphasizing crack control in ice climates. The presence of ice or ice melting chemicals could enter small cracks and cause deterioration of the structure.

Stresses in the beam elements shall not exceed the values given below:

	Tension		Compression
	Metric	English	
1) $S1 = D + L + I + \underset{\substack{\text{or} \\ \text{HF}}}{CF} + LC + LN + LR_1$	0 $250 (f'_c)^{0.5}$	0 $3.0 (f'_c)^{0.5}$	$.45 f_c^3$ $.45 f_c^4$
2) $S2 = D + L + I + \underset{\substack{\text{or} \\ \text{HF}}}{CF} + LC + LN + LR_1$ $+ W_1 + SIV + SIG$	$620 (f'_c)^{0.5}$	$7.5 (f'_c)^{0.5}$	$.60 f_c$
3) $S3 = D + L + I + \underset{\substack{\text{or} \\ \text{HF}}}{CF} + LC + LN$ $+ (LR_1 + RT_1 + T) + W_1$	$620 (f'_c)^{0.5}$	$7.5 (f'_c)^{0.5}$	$.60 f_c$
4) $S4 = D + W_2 + (LR_1 + RT_1 + T)$	$620 (f'_c)^{0.5}$	$7.5 (f'_c)^{0.5}$	$.60 f_c$
f'_c = Concrete compressive strength in Pascals (psi) at 28 days.			

Notes:

1. CF is used in horizontally curved guideway. HF is used in tangent guideway.
2. The critical points on the beam cross sections where the tension stress limit applies is shown below for the two types of cross sections:



These points enclose the primary post-cracking load carrying portions of the beam for resisting shear, torsion, and bending moments. Tensile stresses in the shaded areas of the cross-sections are allowed to exceed the values in the table. Mild steel is placed in these areas to resist the total tensile force in the concrete computed on the assumption of an uncracked section. The stress in this mild steel at service loads shall be such that:

$$f_s (d_c A)^{1/3} \leq 125 \text{ (English Units)}$$

as presented in Section 10.6 of ACI 318-77 (Reference 19). The expression shall be applied to the most severe load condition.

3. For construction joints.
4. For monolithic concrete.

Load Factor Design Criteria:

The basic load factor for dead plus live load is lower than ACI or AASHTO because the loads are much better defined. Physical size restraints on the vehicles and the limitations imposed on hauling cargo prevent over-load conditions as can occur on a highway bridge.

The seismic condition, U_6 , includes the vehicle weight. In actual practice, the contribution of the vehicle will be influenced by the vehicle suspension. Consequently, there is full vehicle weight vertically but only partial weight laterally.

The factored load combinations are:

Ultimate Strength Load Combinations²

- 1) $U_1 = 1.3 D + 2.0 (L + I) + 1.3 CF + 1.3 LC + 1.3 LE$

or
HF
- 2) $U_2 = 1.3 D + 1.5 (L + I + SIV) + 1.3 CF + 1.3 LC + 1.3 LE$

or
HF

 $+ 1.3 W_1 + 1.3 SIG$
- 3) $U_3 = 1.3 D + 1.3 (L + I) + 1.3 CF + 1.3 LC + 1.3 LE$

or
HF

 $+ 1.3 W_1 + 1.0 (R + S) + 1.0 (LR_2 + T)$

or
1.0 (RT₂ + T)
or
1.0 (BR + T)
- 4) $U_4 = 1.0 D + 1.3 W_2 + 1.0 (R + S) + 1.0 (LR_2 + T)$

or
1.0 (RT₂ + T)
or
1.0 (BR + T)
- 5) $U_5^1 = 1.3 D + 1.3 (L + I) + 1.3 LE + 1.3 (COLF_H) + 1.3 W_1$

or
(COLF_V)
- 6) $U_6 = 1.3 D + 1.3 (L + I) + 1.3 LE + 1.0 EQ$
- 7) $U_7 = 1.3 D + 1.3 (L + I) + 1.3 CF + 1.3 LC + 1.3 LE$

+ 1.3 ST
or
HF
- 8) $U_8 = 1.3 D + 1.3 (L + I) + 1.3 CF_2$

Notes:

1. The strength of the beam elements for load case U_5 shall not consider the side walls as part of the load carrying capacity of the beam elements.
2. Load factors will be reviewed upon completion of the ACI Committee 358 report.

The capacity of any concrete member or connection shall be reduced by the capacity reduction factor (ϕ) such that $(U \text{ capacity}) \times (\phi) = U_{load}$.

Stability Considerations:

- 1) Extreme Lateral Force Around Horizontal Curves: The dual-lane structure shall resist overturning due to the cumulative effects of maximum wind on vehicle and structure, maximum centrifugal and wheel curving forces at the wheel/rail contact, gravity load, and extreme operating vehicle live load and impact.
- 2) Derailment: Any portion of the dual-lane structure shall resist overturning due to the eccentric lateral position of a derailed vehicle.
- 3) Wind: Any portion of the dual-lane structure shall resist overturning due to simultaneous wind uplift and wind lateral pressure.
- 4) Column Support Damage:
- 5) Vibration: The intent of the natural frequency limitations is to eliminate the possibility of adverse vehicle-guideway dynamic interaction.

Operational Serviceability:

Adjustability: To the extent possible, all structures and sub-systems shall be designed to allow easy adjustment prior to installation. Once installed, these elements should be firmly fastened so that further undesirable movements are prohibited.

Maintenance: The basic design shall be predicated on a philosophy which minimizes maintenance. However, routine maintenance and inspection of the system shall be done without a major disruption in operations.

Fatigue Criteria:

General:

The fatigue criteria for the elevated guideway structure is derived from References 1, 6. Fatigue criteria are important because many cycles of loading are experienced by ICTS guideway elements.

Service load analysis for fatigue shall be performed on structural elements. Special care shall be exercised on areas of indirect or complicated load paths.

ICTS Guideway Elements:

The basic structural elements in ICTS elevated guideways are beams, crossheads and columns. There are two different types of crossheads. A shallow crosshead occurs at the expansion end of the beams and a full-depth crosshead occurs at the fixed end of the beam. The structural elements resist the fluctuating stresses shown in Table 6.6.1 with each train passage on a given lane of the structure. Table 6.6.1 also lists the constant stress resisted by each element.

<u>Structural Element</u>	<u>PT</u>	<u>Fluctuating Type of Stress</u>	<u>Constant Type of Stress</u>
Beam	A	Live Load + Impact Shear and Torsion	- Dead Load Shear and Torsion
Beam	B	Live Load + Impact Bending	- Dead Load Moment and P/T Moment
Beam	C	Live Load + Impact Bending, Shear, and Torsion	Dead Load Shear, Torsion and P/T Moment
Fixed Crosshead	D	Live Load + Impact Bending, Torsion and Shear	Dead Load Moment and Shear
Fixed Column	E	Live Load + Impact Bending and Axial Load	Dead Load Axial Load
Fixed Column	F	Live Load + Impact Bending and Axial Load	Dead Load Axial Load
Expansion Crosshead	G	Live Load + Impact Shear, Torsion and Moment	Dead Load Moment and Shear
Expansion Column	H	Live Load + Impact Bending and Shear	Dead Load Axial Load
Expansion Column	I	Live Load + Impact Bending and Shear	Dead Load Axial Load

TABLE 6.6.1 FLUCTUATING AND CONSTANT STRESSES
IN ICTS GUIDEWAY ELEMENTS

TRACKWORK DESIGN

This criteria is for the design of the trackwork for the ICTS. The ICTS trackwork will have the following characteristics:

- 1) Composed of continuously welded rail (CWR) to the maximum extent possible.
- 2) Attached by direct fixation to the deck slab of the aerial guideway structure.

This criteria deals with the loadings imposed upon the rail, the requirements of the direct fixation fastener and the identification of design considerations for the aerial guideway structures which are imposed by the trackwork. This criteria is specific to the particular rail system and size.

3. Trackwork Structure Interaction Forces

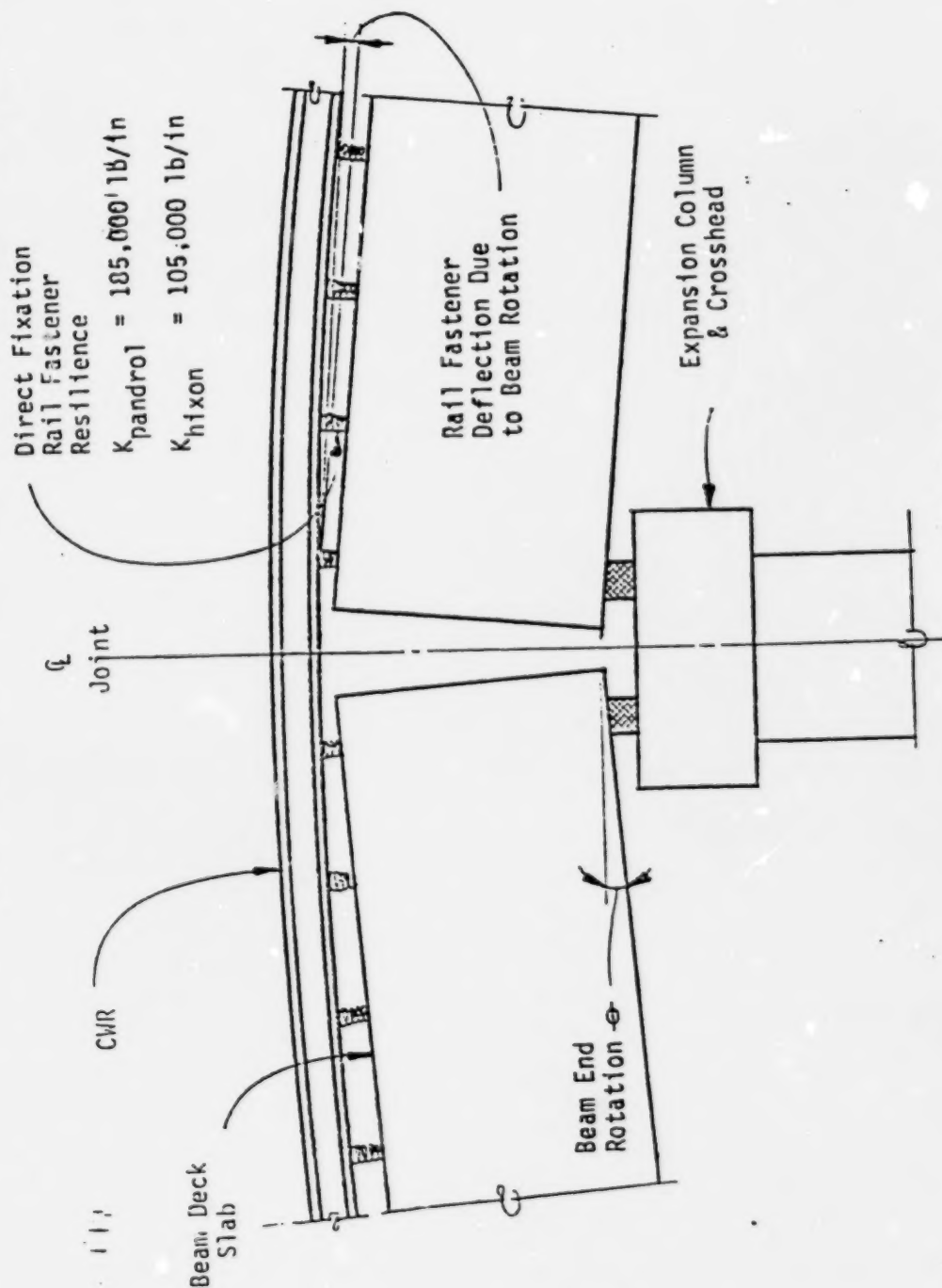
The requirement to consider the interaction between the trackwork and the support structure stems primarily from the fact that the rails are continuous and run over the aerial support structure which is not fully continuous.

The continuous rail, for practical purposes, does not move with variations in temperature while the aerial support structure expands and contracts beneath the stationary rail. This situation results in forces in both the rail and the support structure. See Figure IV-21, p. 55.

At the aerial structure expansion joints the end rotations which result from alternately loading and unloading the beams introduce bending in the continuous rail that crosses the joint. The bending which occurs in the rail at this point is a function of the stiffness of the support structure. See Figure III-3.

The thermal force which is locked in the continuous rail must also be dealt with when the interaction between trackwork and structure is considered. The cases of particular interest are:

- a. Dealing with the radial thermal forces in the continuous welded rail as it negotiates curves.
- b. Dealing with the redistribution of forces that occurs when a continuous rail pulls apart under high thermal tension load.
- c. Dealing with discontinuities in the continuous rail system such as turnouts and rail dead ends.



TRACKWORK STRUCTURE INTERACTION - BEAM END ROTATION

FIGURE III-3

Rail Loadings:

The life of the running rail of the ICTS will be controlled by either rail wear or fatigue. There is no history available to predict the rail wear characteristics with ICTS vehicles. In modern transit systems, curves with radii less than 300 m experience severe wear and require replacement cycles as little as eleven months. Rail wear-prediction models developed for railroads predict similar wear would occur with ICTS loadings and curve geometries.

Thus, until ICTS operational experience is obtained, accurate wear predictions are not possible.

Considerations of rail fatigue will be based upon a life equal to the life of the structure, 50 years. Thus, recurring fatigue failures in the rail will not control rail life.

Thermal forces in the rail result from temperature changes. The magnitude of these thermal forces is discussed below.

<u>Environment</u>	<u>AMBIENT TEMPERATURES</u>		<u>RAIL TEMPERATURES</u>		<u>Design Stress-Free Temp., T_0</u>
	<u>Extreme Minimum</u>	<u>Extreme Maximum</u>	<u>Extreme Minimum</u>	<u>Extreme Maximum</u>	
Group A	-40°C(-40°F)	43°C(110°F)	-40°F(-40°F)	54°C(130°F)	10°C(50°F)

Axial thermal rail stress in the rail is determined by the following equation:

$$f_T = E_R K_R (T_0 - T_R)$$

This produces an axial thermal force in the rail equal to:

$$F_T = E_R K_R (T_0 - T_R) A_R$$

where:

A positive rail stress or force is tensile and a negative rail stress or force is compressive.

f_T = Axial thermal rail stress

F_T = Axial thermal rail force

E_R = Modulus of elasticity of rail

K_R = Coefficient of thermal expansion of rail

T_R = Rail temperature

T_0 = Stress-free temperature of rail

A_R = Cross-sectional area of rail

At horizontal and vertical curves, the axial rail force creates radial rail forces equal to:

$$R_I = F_T / R$$

where:

R_I = Radial thermal rail force, in units of force per length of rail (radially inward for tension in rail, radially outward for compression in rail).

F_T = Axial thermal rail force

R = Radius of rail curvature

All of the above equations apply if there is no motion of the rail. If rail motion is possible, the relaxation of the rail forces must be analyzed. Locations where rail motion will reduce thermal forces include:

- 1) Axial motion of rail at rail expansion joints.
- 2) Radial movements of rail and the supporting structures at curved track.

Direct Fixation Fastener Requirements:

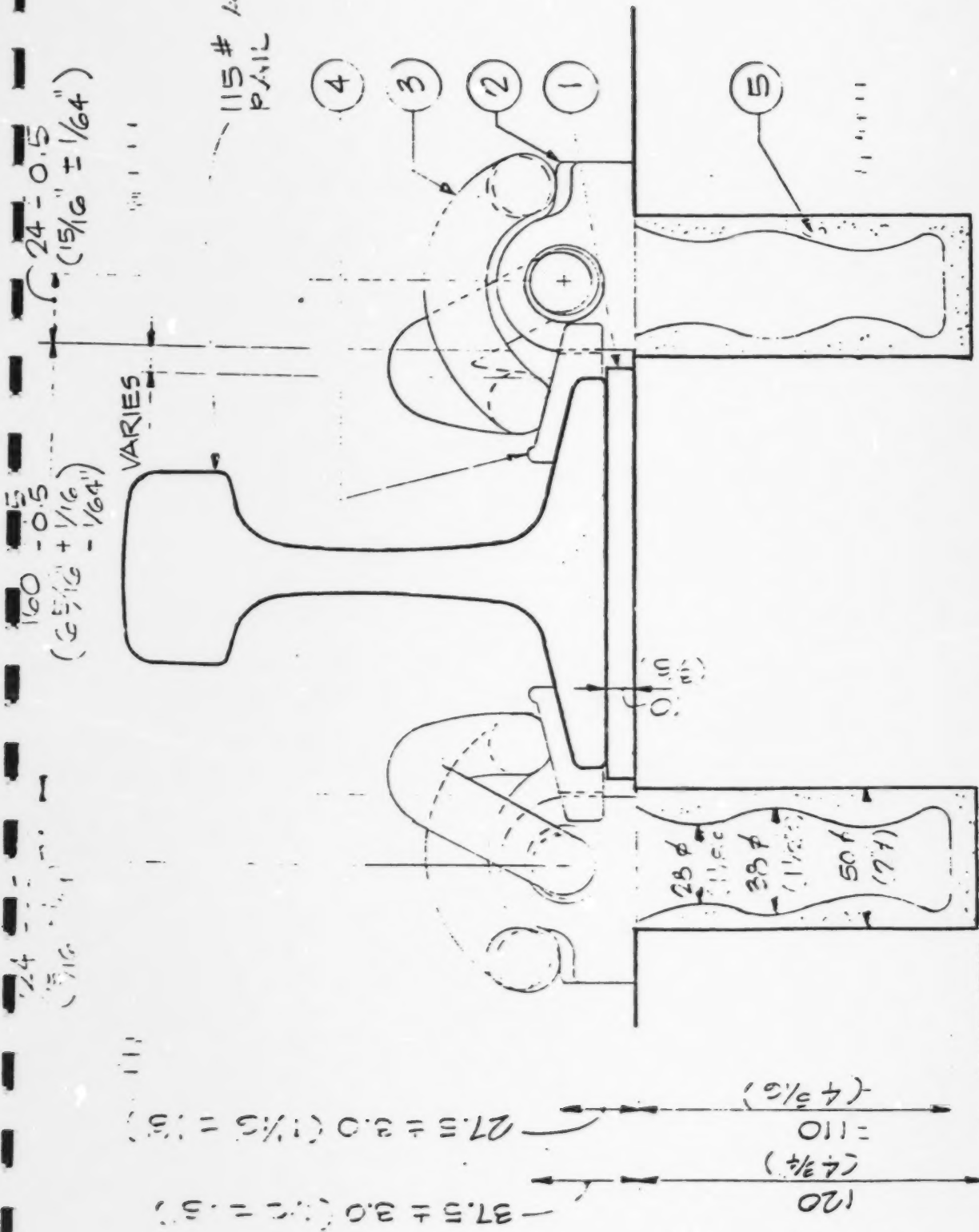
Rail fasteners consist of the hardware which connects the rail to the aerial structure deck slab, including any resilient pads between the rail and structure. Fasteners shall be chosen based upon consideration of the design rail/structure forces, the ability to achieve and maintain desired rail tolerances, and the ability to withstand fatigue and wear of fastener components with reasonable levels of maintenance. It is desirable that the life of the direct fixation fastener approach that of the structure, 50 years.

Pandrol and Hixson fasteners are shown in the following figures.

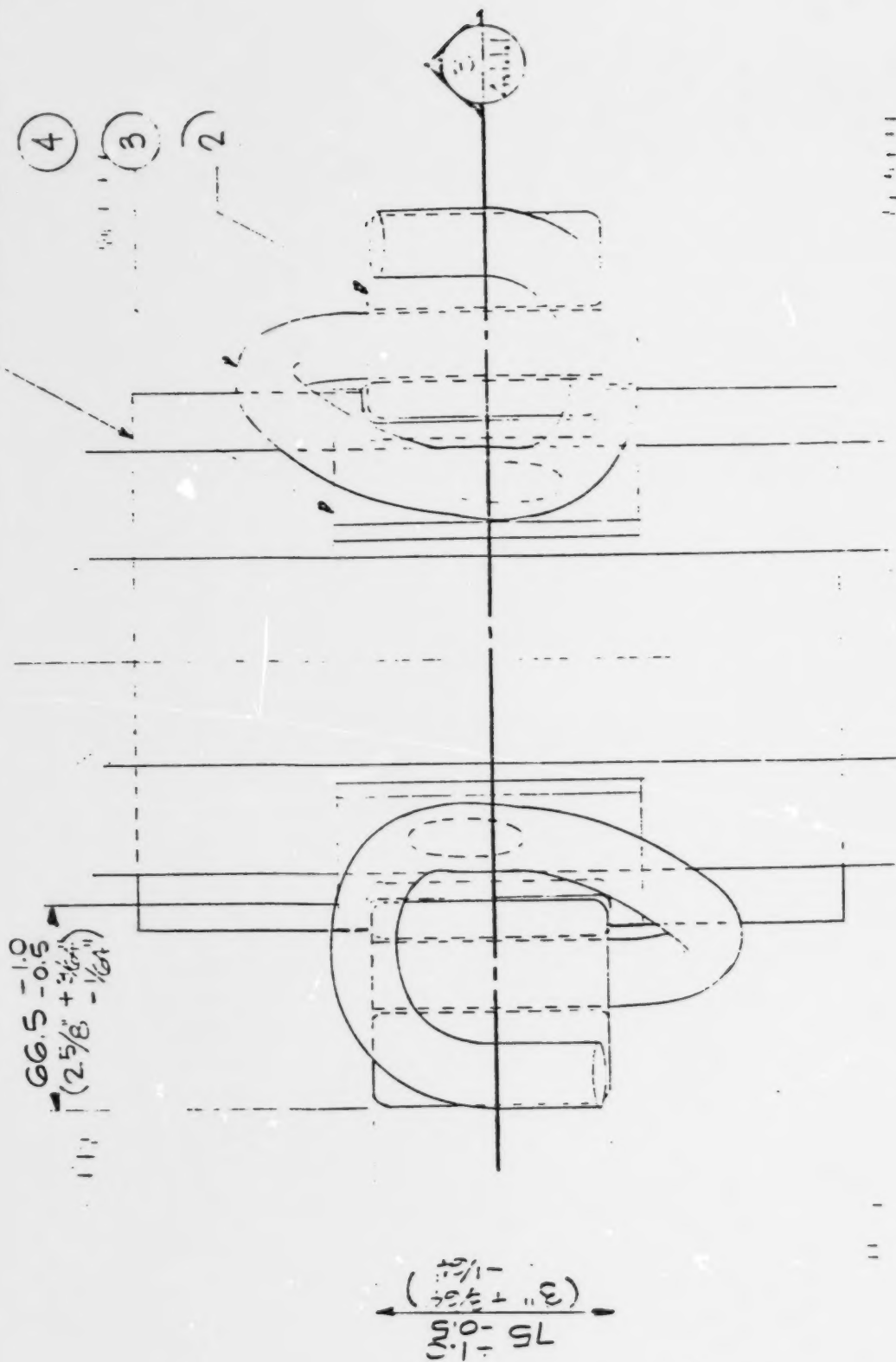
Three million cycles represents less than two years of vehicle wheel load cycles. For expected ICTS applications direct fixation fasteners in similar applications have been in service for less than 10 years. (Landis fasteners at BART have experienced nine years service with little or no degradation).

Analysis of fastener components shall assure that stress levels are within the endurance limits of the material and methods shall be developed to project component wear beyond three million cycles.

For critical fastener requirements of vertical stiffness, gage widening, and longitudinal restraint, refer to Design Criteria and Commentary for the ICTS Guideway Structure. Pull-apart gap, fatigue and wear, and fastener spacing are discussed below.



(3) SECTION 3:2 PANDROL



2 PLAN VIEW
PANDROL

TOP OF RAIL

ASTM A-449
 2.2" ϕ x 100.5 (7'6" x 8'6")
 ANCHOR BOLT ALL
 UNFRADED, NC, 1/2" THICK
 1/2" INCH

2.2" ϕ (3" ϕ) ASTM-449
 LOCKNUT

2.3" x 0.3" x 12.7
 (2" x 1 1/4") WASHER

RAIL CLIP SHIM (8)
 (2400, LRC ONLY)
 RAIL CLIP (6) OR (7)

19.0" ϕ (3 1/4" ϕ)
 STOVER LOCKNUT

19.0" ϕ x 69.3 (3'6" x 5'8")
 CARRIAGE BOLT

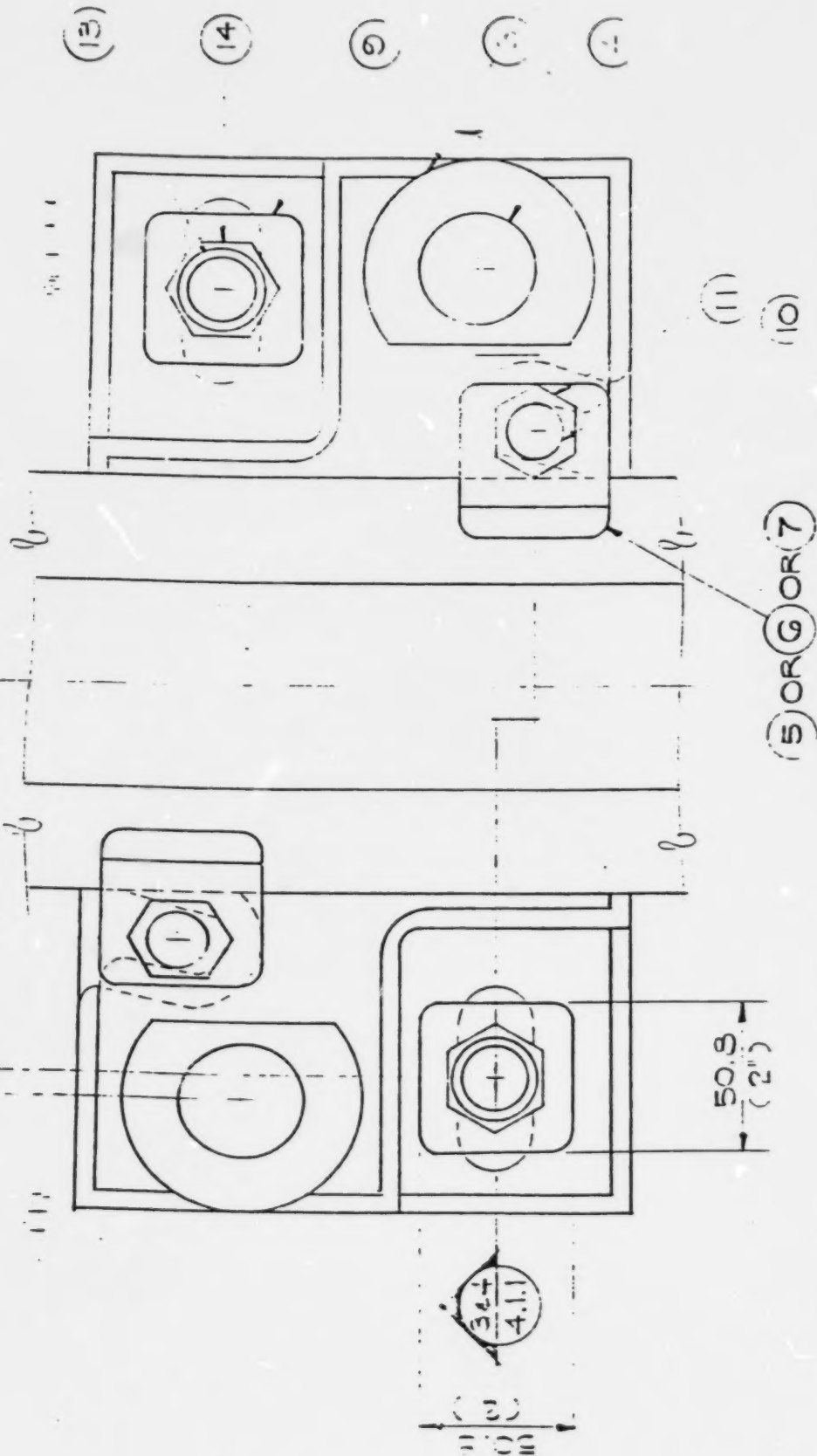
HOLD DOWN
 DEVICE

(6)
 4.1.1

10mm (3/8") RAIL SHIM, SEE DETAIL

SECTION HIXSON

112
 TYPICAL 6000, 2400 TYPE



2) POLYNOMIAL HIXSON

二十一

Pull-Apart Gap:

The longitudinal restraint of the fastener shall be such that the calculated gap at a pull-apart will be less than 50 mm (2.0 in) at the minimum expected rail temperature. Computation of this gap shall include both rail slip in the fasteners and deflection of the aerial structure.

Fastener Spacing:

The spacing of direct fixation fasteners will be determined after the investigation of the following considerations:

- 1) Rail buckling.
- 2) Gage widening.
- 3) Longitudinal restraint.
- 4) Rail stresses.
- 5) Fastener loadings.
- 6) Rail wear considerations.
- 7) Other considerations.

Rail Loadings Applied to Aerial Structure:

Longitudinal Restraint Forces (LR_1 , LR_2) on Tangent Guideway:

The incompatibility of motions between tangent structures and the continuous rail cause shears in the direct fixation fasteners which apply axial forces and uniformly distributed couples to the beams. Fastener restraints may increase due to malfunction, thus increasing loads applied to the structure.

Normal operating fastener restraint, LR_1 , shall apply to service load design while the malfunction restraint, LR_2 , shall apply to strength design.

LR_1 and LR_2 are as shown below:

Environment Group	Value of Fastener Longitudinal Restraint	
	Newton (LBS) LR_1	Newton (LBS) LR_2
A	13,340 (3000)	20,000 (4500)

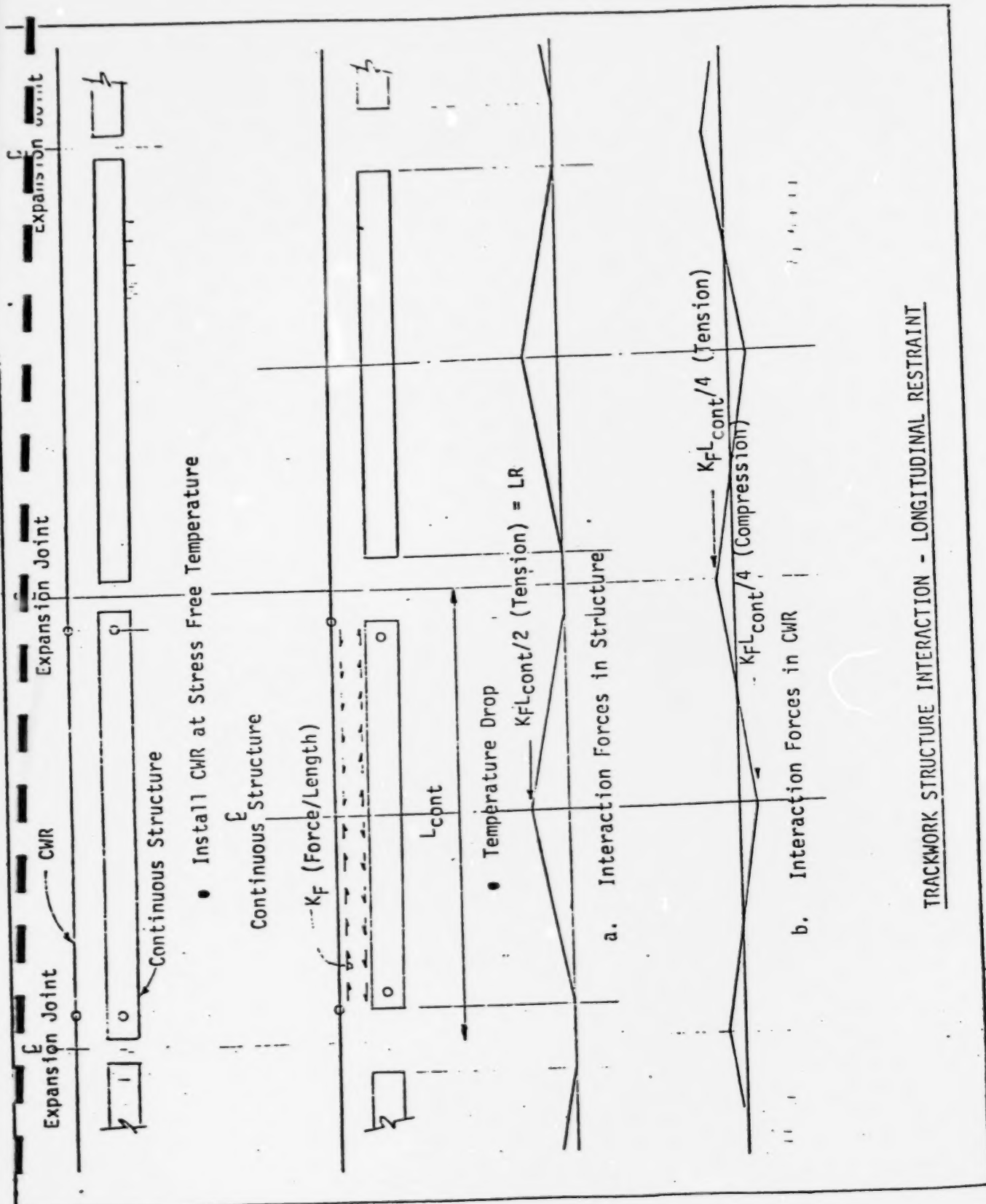
Fastener Spacing:

<u>Tangent track</u>	<u>Curved track</u>
760 mm (30 in.)	610 mm (24 in.)

Figure IV-21a shows that the magnitude of this force in the structure is dependent on the longitudinal fastener restraint and the length of continuous guideway structure. It should be noted that this force results when any motion of the bridge occurs since a very small slip mobilizes the longitudinal fastener restraint. Thus, it does not only occur when the structure is very hot or very cold. For this reason, the normal or tested value of the longitudinal fastener restraint is imposed on the basic service load design condition. This force is shown as LR_1 . As previously discussed, the malfunction restraint force is the normal restraint force multiplied by 1.5. This force, shown as LR_2 , is included in ultimate load design cases U_3 and U_4 .

Figure IV-21b shows the forces in the rail created by this motion. The magnitude of the force in the rail is maximum at the structure expansion joint and minimum at the point of structure zero motion. The average restraint induced force in the rail between structure expansion joint is zero since the rail does not change length between structure expansion joints.

Examples of calculations of these forces for the ICTS elevated structure are shown in Appendix B. The calculations are based on earlier design criteria which may have been revised.



TRACKWORK STRUCTURE INTERACTION - LONGITUDINAL RESTRAINT

Unbalanced Thermal Forces:

When an unbalance of rail thermal forces exists the aerial structure must be designed to resist this unbalanced force. These unbalanced forces result in the following conditions:

- 1) In curve transition structures due to thermal force relaxation due to radial deflections in curved structures.
- 2) Where adjacent structures have non-symmetrical fastener longitudinal restraint characteristics or non-symmetric structural configurations.
- 3) At rail joints.
- 4) At abutments.
- 5) At rail anchors.
- 6) At turnouts.

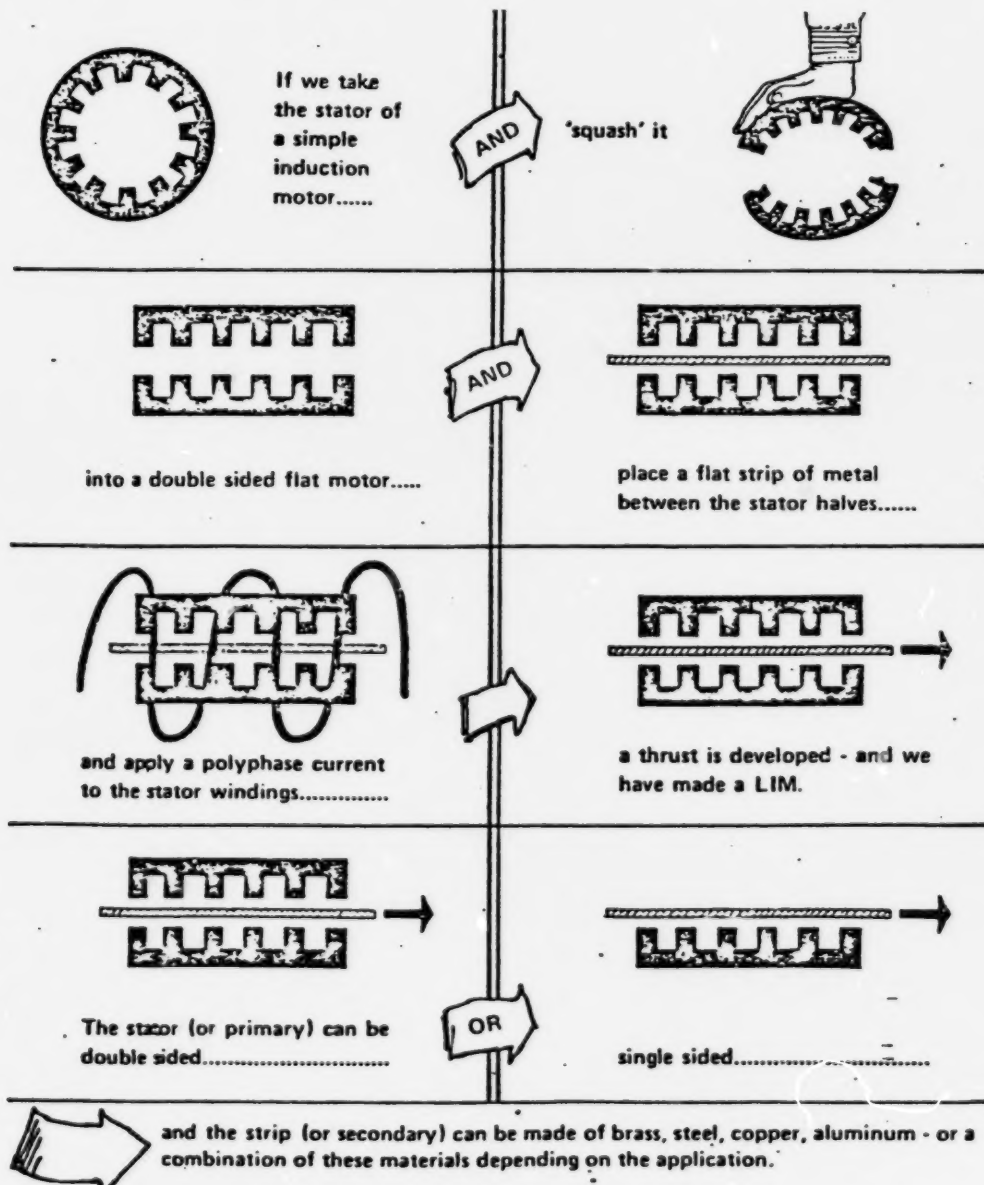
The unbalanced forces will be determined for each of these cases.

General:

The relationship between a linear motor and a rotary motor is graphically illustrated below.

Unlike rotary motors, linear motors do not require gears and transmissions and are consequently more simple and rugged.

The thrust of a low-speed linear induction motor is proportional to the square of the applied voltage and drops to zero at synchronous velocity. For best performance, the secondary should be an aluminum plate backed with mild steel or iron. The normal or attractive force between primary and secondary is dependent on the secondary construction. The air gap between the LIM primary and secondary should be as small as possible, consistent with mechanical clearance requirements. In most applications speed is controlled using a velocity feed back signal to modulate the input voltage to the motor.



The LIM motor is used for the ICTS propulsion system to provide for rapid smooth longitudinal acceleration and deceleration. This allows closer vehicle headways. Use of the LIM also removes the primary braking mode of the vehicle from the wheel rail contact surface. This allows the vehicle to operate on a harsh snow and ice environment.

The LIM consists of the LIM primary rigidly fixed to the axles of the vehicle (two per vehicle) and the LIM secondary attached to the guideway. The LIM secondary consists of an aluminum top cap placed over a steel core. Two configurations of this steel core are used, a laminated back iron and a solid back iron. The laminated back iron is more costly and is used in areas where higher performance is desired.

LIM Tolerances:

The air gap between the LIM secondary which is attached to the guideway and the LIM primary which is attached to the vehicle has a direct effect on the acceleration performance of the vehicle. The LIM secondary must be adjusted in the vertical direction to ± 6 mm. This adjustability is required for initial installation and to account for rail head wear during the life of the installation.

POWER RAIL DESIGN PARAMETERS

The power rail shall be mounted from the concrete wall on either side of the guideway. The power rail feeds the vehicle LIM motors through a two-rail, 600 v direct current system. Sets of carbon collector shoes are mounted at each end of the vehicle.

Service Life:

The design life of the power rail bracket on the mainline shall be 4.38×10^7 load cycles from each set of collector shoes mounted.

Thermal Loads:

The power rail design shall consider thermal strains due to temperature changes in the rail.

The rail may be continuously attached to the guideway structure as is the running rail. In this case, the power rail must be capable of withstanding the force developed by not allowing thermal movements. Other structure/power rail interactions must be considered.

The rail may be allowed to slide at discrete locations. The locations of these power rail expansion joints shall consider the forces and motions developed by structure movements.

The temperature range of the power rail shall be determined for Environment A and shall consider maximum and minimum ambient temperatures, solar radiation and heating due to electrical resistance.

TOLERANCES:

GENERAL

The determination of target tolerances for various ICTS subsystems was important.

A reasonable track tolerance was required to accommodate vehicle suspension and guidance system design while not restricting the required ride comfort.

The LIM installation tolerance has a significant effect on the required air gap between the LIM secondary mounted on the guideway and the vehicle-mounted LIM primary.

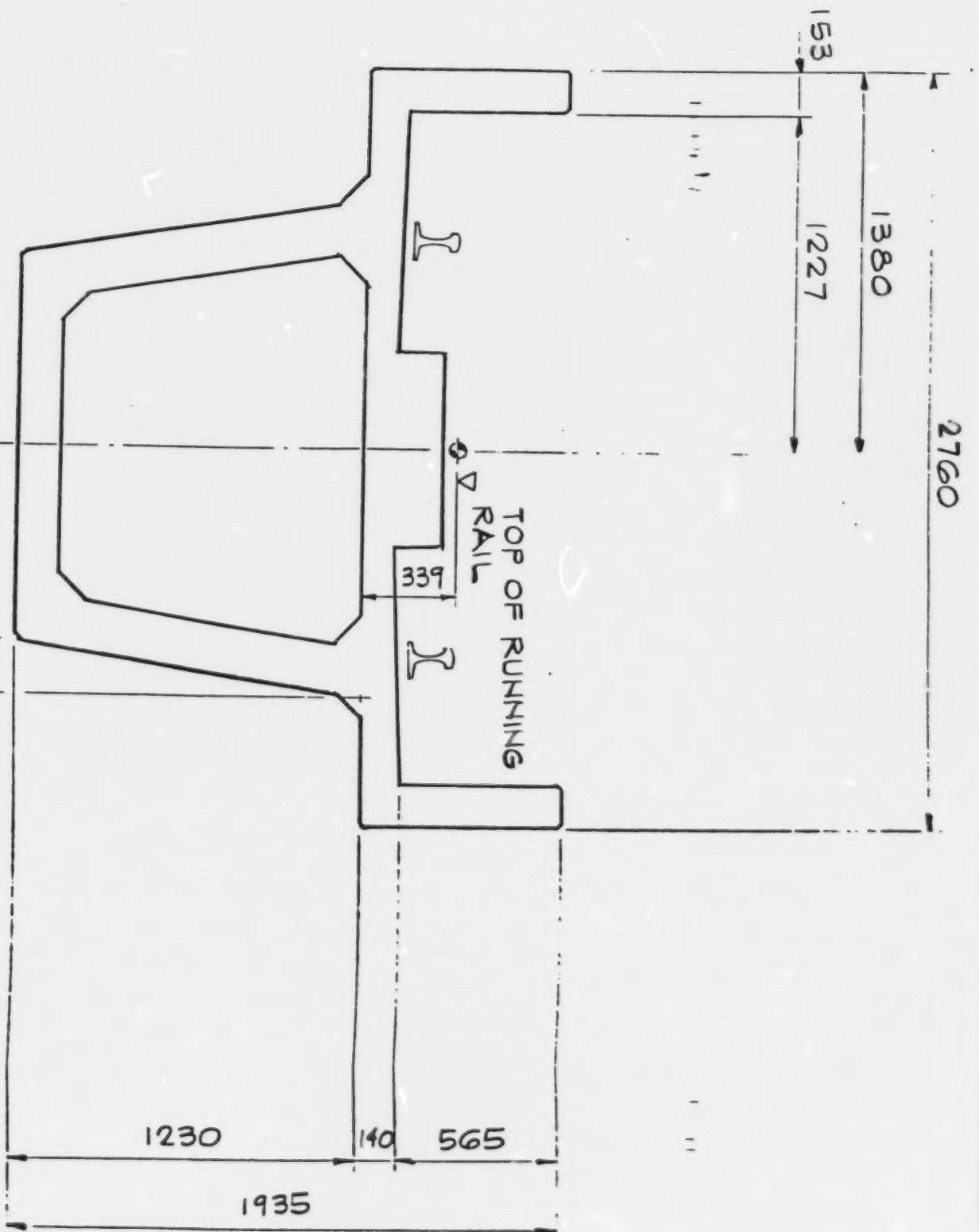
Power rail mounting provisions are required to accommodate the design of the vehicle-mounted power pickup.

During construction of the ICTS test track construction methods were modified to achieve increasingly more precise tolerances. Rigorous inspection and correction is required at each level of construction in order to achieve the desired precision in the rail installation tolerances.

Achievement of gage and alignment tolerances for curved rails is largely dependent upon the quality control of the turnout fabrication. The use of base plates with lateral and vertical adjustment capabilities would increase the tolerance achievement level.

In order to achieve progressively closer levels of precision in construction of the at-grade slab a thorough program of survey control was planned. The reinforcing was installed to assure non-interference with the track fastener hole drilling operations. Finishing of the slab surface was performed by three machines. Shimming and grinding were employed to achieve the desired rail profile.

The construction of the cast-in-place, post-tensioned, elevated guideway was based on the same system of survey controls as was the at-grade slab.



Tolerances for the concrete structure are as given in the "Canadian Prestress Concrete Institute Handbook for Commercial Grade" in addition to the following:

Beams:

- (a) Longitudinal surface: $\pm 1/8$ inch in 10 feet.
- (b) Transverse surface: $\pm 1/8$ inch in 10 feet.
- (c) Deviation from specified camber: $\pm 1/8$ inch in 10 feet.

Walls:

- (a) Tolerance on the vertical walls longitudinal surfaces including tops: $\pm 1/4$ inch in 10 feet.
- (b) Vertical wall thickness: $-1/8$ inch to $+1/4$ inch.
- (c) Vertical walls out of plumb: $1/8$ inch per foot of height but not to exceed $3/8$ inch.
- (d) Height of wall: $\pm 3/4$ inch with respect to the top of rail.

Columns: Construction tolerances for columns shall be:

- (a) Tolerances of the column cross-section: $-1/8$ inch to $\pm 1/4$ inch.
- (b) Surface smoothness: $\pm 1/8$ inch in 10 feet.
- (c) Plumbness: $1/4$ inch in 8 feet but not to exceed $1/2$ inch.

TRACKWORK TOLERANCES

Target tolerances for construction of the ICTS trackwork are shown in Figure V- 23. Target tolerance goals for special trackwork are shown in Figure V-35.

Tolerance	TARGET SPECIFICATIONS FRA Class 6 Canada 1 ^r		ICTS MEASUREMENTS 95%		CN MAINLINE ^A 95%	
			100%			
1) Alignment: Deviation from design offset at the mid-ordinate of a 62' chord	Curved $\pm 3/8"$	$\pm 1/8"$	$\pm 1/8"$	$\pm 15/64"$	$\pm 5/16"$	$\pm 1/2"$
	Tangent $\pm 1/2"$					
	Curved $\pm 9 \ 1/2 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 6 \text{ mm}$	$\pm 8 \text{ mm}$	$\pm 12 \text{ mm}$
	Tangent $\pm 12 \ 1/2 \text{ mm}$					
2) Profile: Deviation design offset at the mid-ordinate of a 62' chord	$\pm 1/2"$	$\pm 1/8"$	$\pm 3/32"$	$\pm 13/64"$	$\pm 11/16"$	$\pm 7/8"$
	$\pm 12 \ 1/2 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 2 \ 1/2 \text{ mm}$	$\pm 5 \text{ mm}$	$\pm 10 \text{ mm}$	$\pm 21 \text{ mm}$
3) Deviation from design gage	Tangent $-1/2"$, $+1/4"$	$\pm 1/8"$	$\pm 1/8"$	$\pm 9/32"$	$\pm 3/16"$	$\pm 1/4"$ (95)
	Curved $\pm 1/2"$					$-1/2"$
	Tangent $-12 \ 1/2 \text{ mm}$, $+6 \ 1/2 \text{ mm}$					$+6 \text{ mm}$ (95)
	Curved $\pm 12 \ 1/2 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 7 \text{ mm}$	$\pm 5 \text{ mm}$	-13 mm
4) Deviation from design cross level	$\pm 1/2"$	$\pm 1/8"$	$\pm 1/8"$	$\pm 7/32"$	$\pm 3/8"$	$\pm 9/16"$
	$\pm 12 \ 1/2 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 3 \text{ mm}$	$\pm 5 \ 1/2 \text{ mm}$	$\pm 10 \text{ mm}$	$\pm 15 \text{ mm}$
5) Change in deviation from design cross level between any two points less than 62' apart	$5/8"$	$15/64"$	$13/64"$	$21/64"$	$\pm 3/8"$	$\pm 1/2"$
	16 mm	6 mm	5 mm	$8 \ 1/2 \text{ mm}$	$\pm 10 \text{ mm}$	$\pm 13 \text{ mm}$

^A 3000 ft (914 m) length of CN Mainline Track near Montreal measured to give background data.

RAIL TOLERANCE SPECIFICATION AND ACHIEVED AS-BUILT TOLERANCE LEVELS

Figure V-23

TARGET TOLERANCE GOALS FOR SPECIAL TRACKWORK

Description of Feature	Tolerance	
	Metric	English
Lateral location of straight stock rail	± 3 mm	($\pm 1/8$ in.)
Straightness of straight stock rail	± 3 mm in 10 m	($\pm 1/8$ in. in 32.8 ft.)
Offset between gage faces of straight stock rail and curved switch/closure rail	± 3 mm	($\pm 1/8$ in.)
Gage	± 3 mm	($\pm 1/8$ in.)
Rail elevation	± 3 mm	($\pm 1/8$ in.)
Rail profile smoothness	± 3 mm in 20 m	($\pm 1/8$ in. in 65.6 ft.)
Cross level	± 3 mm	($\pm 1/8$ in.)
Cross level variation	± 3 mm in 10 m	($\pm 1/8$ in. in 32.8 ft.)

FIGURE V-35

Appendix A

REFERENCES

CODES AND REFERENCESRelated Transit Codes:

Relevant portions of the following transit codes and criteria have been given close consideration in the formulation of the Design Criteria and Commentary for the ICTS guideway structure:

- 1) AASHTO Standard Specifications for Highway Bridges, 1977. (American Association of State Highway and Transportation Officials.)
- 2) MTC Ontario Highway Bridge Design Code, 1979. Ministry of Transportation and Communications.
- 3) CSA-S6 Design of Highway Bridge (Canadian Standards Association), 1974.
- 4) AREA Railway Design Code (American Railway Engineering Association), 1973.
- 5) CSA-S1 Steel Railway Bridge (Canadian Standards Association), 1966.
- 6) ACI 443 Analyses and Design of Reinforced Concrete Bridge Structures, 1977.
- 7) BS-116 British Standards Institute Technical Committee, Steel and Concrete Bridges (Draft), 1976.
- 8) BART Bay Area Rapid Transit Design Criteria, 1963.
- 9) MARTA Metropolitan Atlanta Rapid Transit Authority Design Criteria, 1974.
- 10) WMATA Washington Metropolitan Area Transit Authority Design Criteria, 1967.
- 11) Detroit Downtown People Mover Performance Specifications, 1980.
- 12) Los Angeles Downtown People Mover System Specifications, April 1980.
- 13) Miami Downtown People Mover System Specifications, August 1980.
- 14) CALTRANS California Department of Transportation, Bridge Planning and Design Manual, April 1977, Section 2-16, Seismic Forces.

Criteria for the elevated portion of guideway are based primarily on the ACI Committee 443 report (Reference 6). This report is the most

current data available that addresses fatigue and serviceability for repeated loads. ACI Committee 443 is the bridge code equivalent of Committee 318 for building codes.

Criteria for the at-grade portion of the guideway are based primarily on the U.S. Steel "Design Manual for Continuously Reinforced Concrete Pavement" (Reference 27), as listed in the following section. It also draws from past experience in the design and construction of at-grade systems, since the design effort demands a much more empirical approach than that for elevated systems. This is due to the heavy emphasis on soil/structure interaction.

3.2 Reference Documents:

Other codes, recommendations and papers have been instrumental in the development of this Design Criteria and Commentary:

- 15) "Review of Guideway Design Criteria in Existing Transit System Codes," R.A. Dorton, and H. Grouni, Ontario Ministry of Transportation and Communications, November 1975.
- 16) Canadian National Building Code (NBC).
- 17) Climatic Information for Building Design in Canada, Supplement No. 1 to NBC, 1975.
- 18) Wind Forces on Structures. (A compilation of six papers presented in the July 1958 ASCE Journal of the Structural Division.)
- 19) Building Code Requirements for Reinforced Concrete (ACI 318-77).
- 20) Code for the Design of Concrete Structures for Buildings (CSA-A23.3).
- 21) Concrete Materials and Methods of Concrete Construction (CSA-A23.1).
- 22) Manual of Quality Control (PCI Manual 116).
- 23) Billet-Steel Bars for Concrete Reinforcement (CSA-G30.5).
- 24) Welded Steel Wire Fabric for Concrete Reinforcement (CSA-G30.5).
- 25) Tentative Recommendations for Prestressed Concrete, (ACI-ASCE Joint Committee 423).

- 26) The Ontario Ministry of Transportation and Communications. General Conditions and Standard Specifications (Volumes 1 and 2).
- 27) Design Manual for Continuously Reinforced Concrete Pavement (United States Steel Corporation), 1970.
- 28) National Building Code of Canada, Supplement No. 4 Canadian Structural Design Manual, 1970.
- 29) "Recommended Lateral Force Requirements and Commentary," Seismology Committee, Structural Engineers Association of California, 1975.
- 30) "The Steady-State Aerodynamics of Elevated Rectangular Transit Vehicles," Teunissen and Cooper, Laboratory Technical Report No. LTR-LA-179, National Research Council of Canada, 1975.
- 31) "Thermal Expansion of Concrete and Reinforced Concrete," Berwanger and Sarkar, Journal of the American Concrete Institute, November 1976.
- 32) Seismic Design of Highway Bridges, Workshop Manual, February 5-9, 1979, Olympia, Washington. Sponsored by USDOT (FHA) and given by Engineering Computer Corporation (ECC), Sacramento, California.
- 33) Rational for Determining Design Wind Velocities, A.G. Davenport, ASCE Transaction, 1961, Part II.
- 34) Distributions of Extreme Winds in the United States, H.C.S. Thom, ASCE Transaction, 1961, Part II.

Portions of the Design Criteria, particularly specific data, are based on the following ICTS documents:

Appendix B

SAMPLE CALCULATIONS OF TRACKWORK FORCES

NOTE: Calculations in this section are based on earlier design criteria the values of which may have changed.

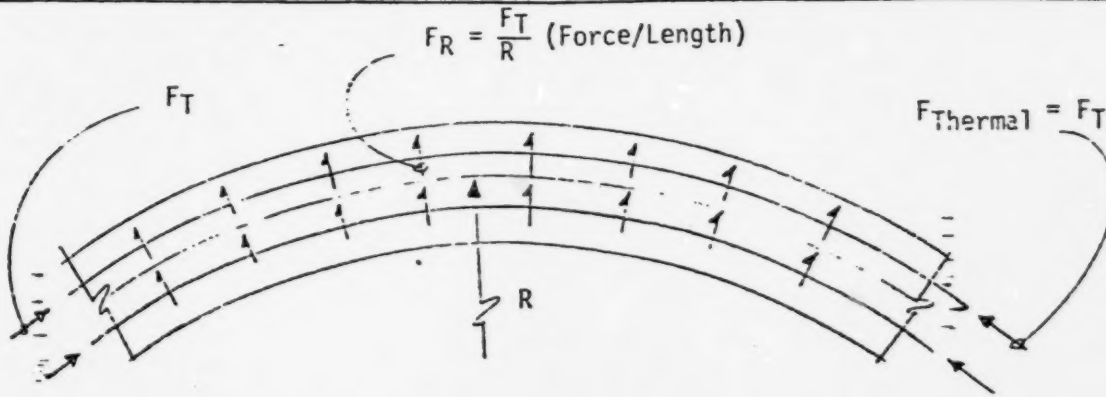
b. Radial Rail Forces on Curves:

Continuous welded rail laid on curves imposes forces which are radial to the curve onto the tie down fasteners and the supporting structure. The magnitude of this force per foot per rail is the magnitude of the thermal force in the rail divided by the radius of curvature.

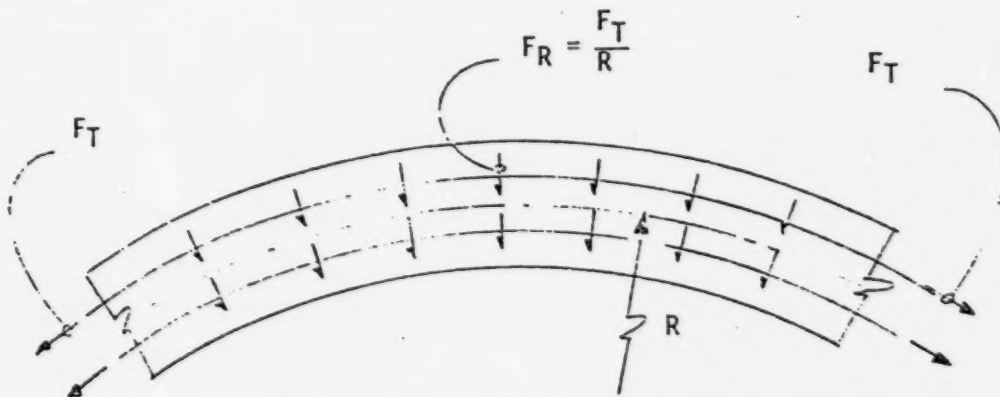
The orientation of this force is shown in Figure IV-22.

As the curve radius decreases, the radial force has an increasingly significant effect on the guideway beam and column design. The effect of this force on the elevated guideway structure is to apply torsion, lateral bending and lateral shears to the beam. The most significant effect is the large lateral load this effect applies at the top of the columns. This load places a large bending moment at the base of the column and a large eccentricity on the foundation.

These moments and eccentricities are doubled for dual-lane guideway construction since a single column restrains the load from four rails instead of two rails as for a single-lane structure.



a. Rail Temperatures > Stress Free Temperature



b. Rail Temperatures < Stress Free Temperatures

RADIAL THERMAL FORCES

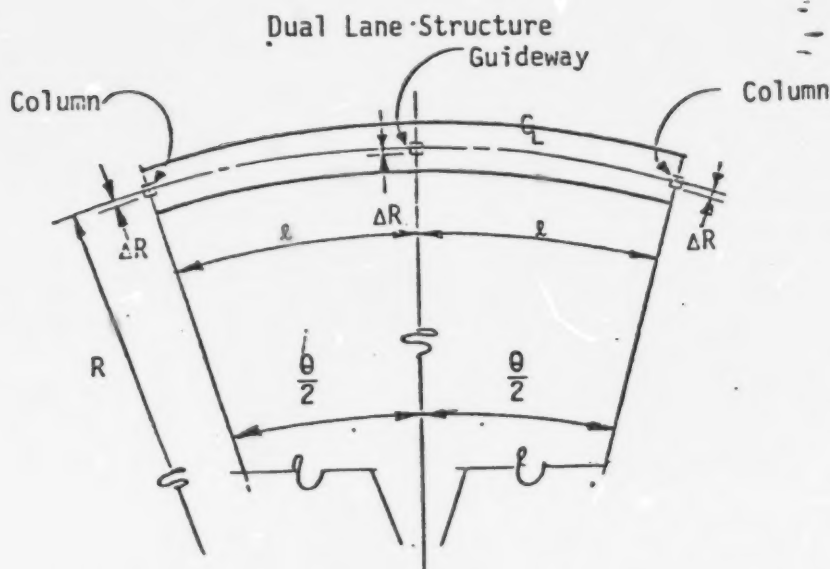
The total radial rail force is never imposed on the structure since the rail is allowed to move radially somewhat by the gap between the rail and the lateral restraining clip and by the side sway motion of the curved structure. For certain horizontal geometries and structure configurations, the radial rail force seems to become almost insignificant. Example IV-3 demonstrates how the radial rail force in curves is reduced. In essence, the thermal rail force is decreased in negotiating curves to the values computed in Example IV-3. The force in the rail must vary between the tangent structure and the curved structure (see Figure IV-23). The length of rail over which the transition occurs is proportional to the longitudinal restraint capability of the fasteners. This then indicates that the structure over which the buildup of rail force occurs must be capable of *opposing* a longitudinal unbalance in force. The longitudinal motion in turn increases the strain in the curved rail adjacent to the transition structure.

Therefore, this phenomenon of a reduction in radial rail force and the force transition length is one that deserves more attention and verification by experiment or full-scale field tests.

Note: This example has been shown to be invalid for one geometry.
A computer analysis of this case should be performed.

EXAMPLE IV-3

RELAXATION OF RAIL CURVATURE FORCES IN COLUMNS



If the rail is fully restrained during a temperature change ΔT , the strain in the rail $= \alpha \Delta T$, where α is the thermal coefficient of expansion for the rail.

If the column deflects ΔR radially due to the rail thermal curvature force, the strain in the rail $= \alpha \Delta T - \frac{\Delta R}{R}$ Equation A

Additionally, the rail is allowed to move radially relative to the structure within the fastener an amount ΔF , due to fastener tolerance and the fastener deformation.

An approximate value of $\Delta F = 0.010$ ft. (3 mm).

Therefore, the net rail strain $= \alpha \Delta T - \frac{\Delta R}{R} - \frac{\Delta F}{R}$ and

$$F_R = \left(\alpha \Delta T - \frac{\Delta R}{R} - \frac{\Delta F}{R} \right) E_R A_R, \quad \text{Equation B}$$

where: F_R = The longitudinal thermal force in the rail.

$$RT = \frac{F_R}{R} = \left(\alpha \Delta T - \frac{\Delta R}{R} - \frac{\Delta F}{R} \right) E_R A_R \times \frac{1}{R} \times 4 \times \frac{R\theta}{1.6} \quad \text{Equation C}$$

where: R_T = Radial thermal force applied to center column
 4 = Number of rails

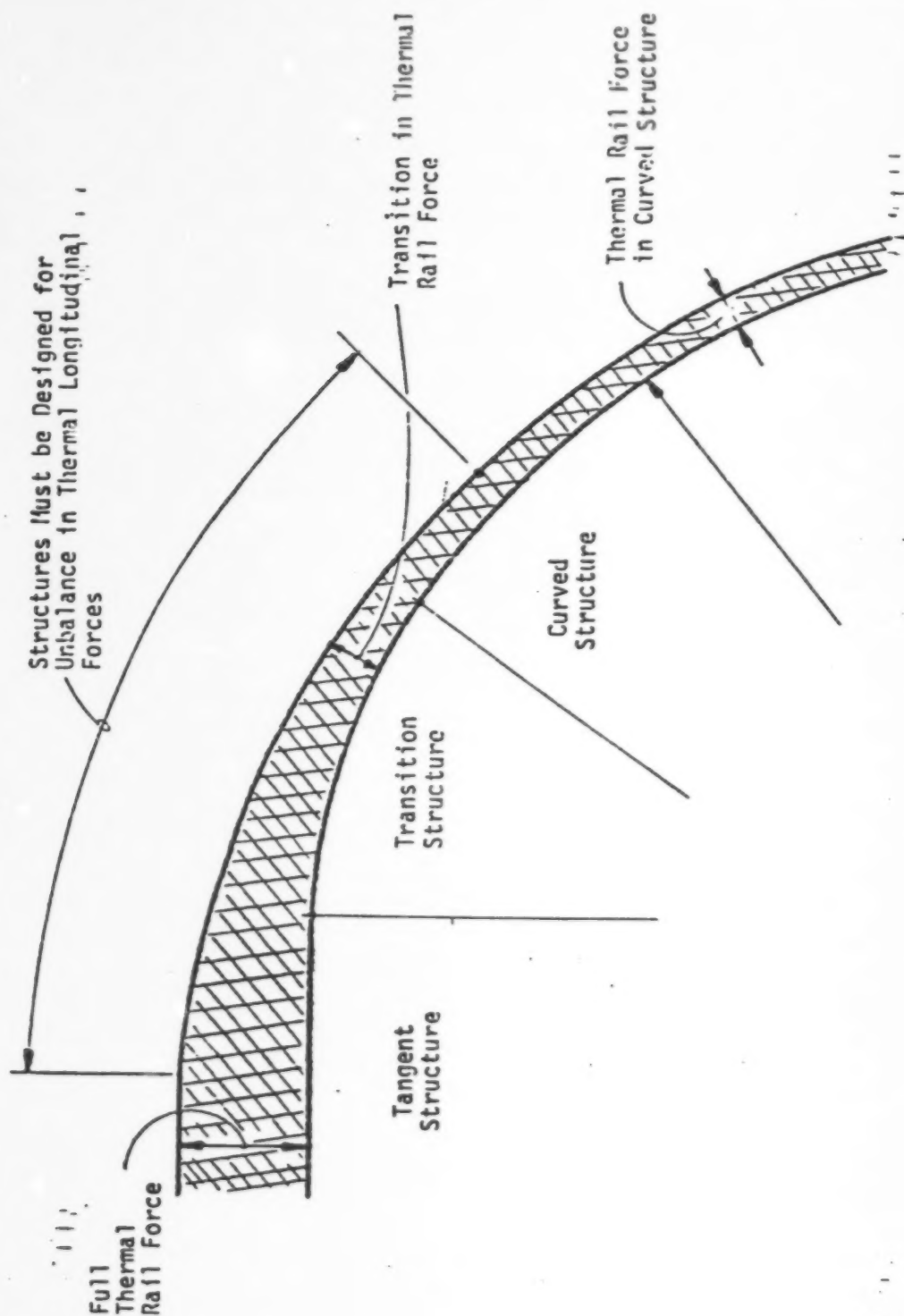
$\frac{R\theta}{1.6}$ = Tributary length of structure for radial rail forces
to center column.

also: $R_T = \Delta R K_{col}$

Equation D

where: K_{col} = Column stiffness in lateral direction.

Combining equations C and D, $\Delta R = \frac{2.5 \theta E_R A_R (\alpha \Delta T - \frac{\Delta F}{R})}{K_{col} + \frac{2.5 \theta E_R A_R}{R}}$



TRACKWORK STRUCTURE INTERACTION -
Thermal Rail Force
Variation in Curved Structures

FIGURE IV-23

c. Broken Rail Force:

It is likely that a nominal number of rail pull-aparts will occur during the expected rail life. Therefore, the effects of a broken rail are considered in the design of the guideway structure. The probability that more than one rail will break at the same time on a particular structure was thought to be small and thus was not considered in the design.

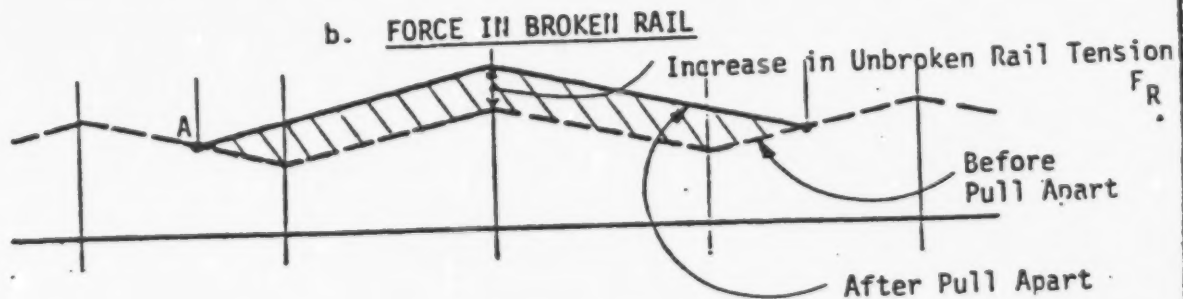
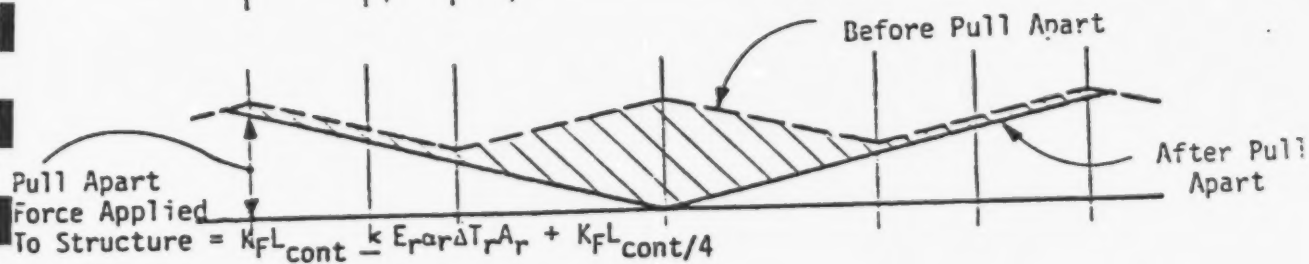
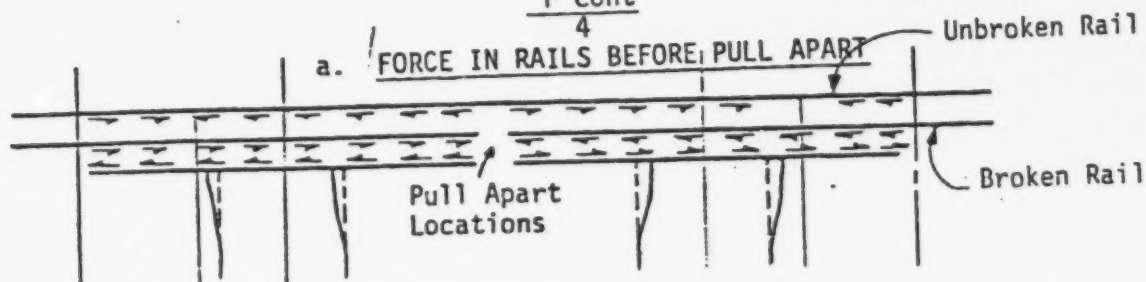
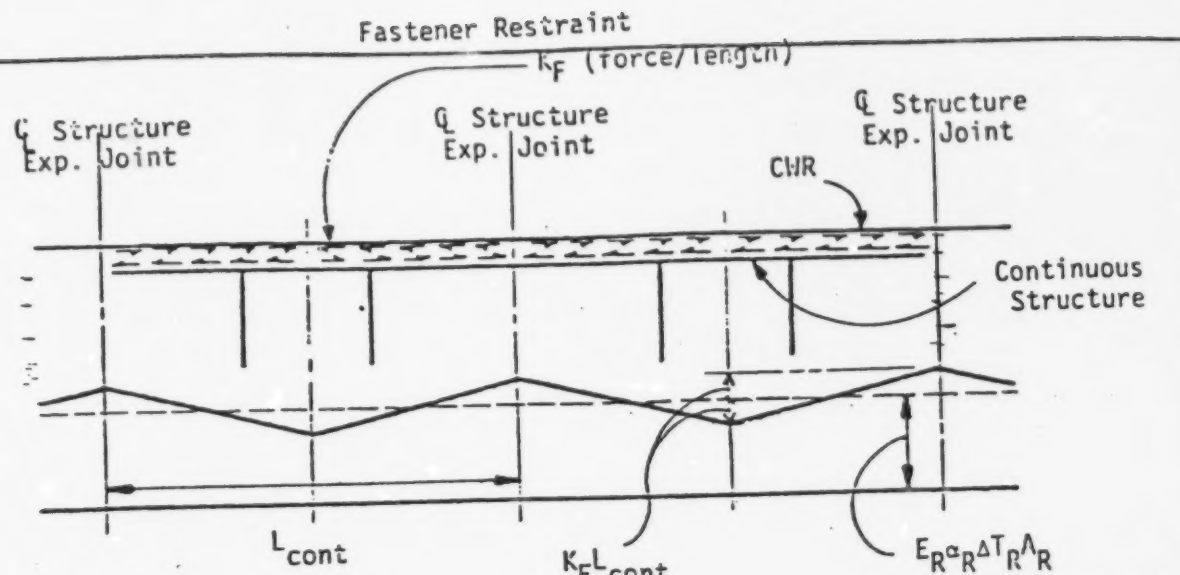
The most likely place for a rail pull-apart to occur is at a weld or defect in the rail near a structure expansion joint. It is at this location that the rail stresses are highest due to the cycle of live load rotation and longitudinal restraint. Additionally, the probability of a rail pull-apart increases as the rail temperature drops increasing the tensile force in the rail.

The rail pull-apart problem has been examined analytically. Whereas, the analysis appears to be sound, a test verification of the analysis would be most beneficial.

The effects of a broken rail are shown in Figure IV-24.

Prior to a rail pull-apart a force equal to the thermal force plus the longitudinal restraint force exists in both rails at the structure expansion joint. When a rail breaks, mathematically a force equal to this force is applied to the broken rail.

The resulting longitudinal compression force needed to react the force released by the broken rail is shared by the unbroken rail and by the guideway structure in proportion to their respective longitudinal stiffnesses.



c. FORCE IN UNBROKEN RAIL

BR = Force Applied to Columns = Force Applied to Structure
Less Force Resisted by Unbroken Rail, F_R .

TRACKWORK STRUCTURE INTERACTION BROKEN RAIL EFFECTS

FIGURE IV-24

This stiffness of the unbroken rail is found from examining Figure IV-24c. The additional force in the rail is constant until we get to the point where the rail no longer slips through the fastener or where the new force curve with a slope equal to the fastener restraint per foot intersects the force curve prior to a rail break. This analysis is very similar to the analysis of computing the effect of a seating loss in a post-tensioning tendon.

The elongation of the rail between Point A and the expansion joint at the rail pull-apart equals:

$$\Delta_R = \frac{\frac{F_R^2}{4k_F} + \frac{F_R L_{\text{cont}}}{A_R E_R}}{F_R} \quad \text{Equation 1}$$

$$K_R = \frac{F_R}{\Delta_R} = \frac{A_R E_R}{(L_{\text{cont}}/2) + \frac{R}{4k_F}} \quad \text{Equation 2}$$

Δ_R = Elongation of the rail between Point A and the expansion joint at the rail pull-apart.

K_R = Longitudinal stiffness of unbroken rail

K_F = Longitudinal fastener restraint

F_R = Increase in unbroken rail tension at a rail pull-apart

A_R = Cross sectional area of rail

E_R = Modulus of elasticity of rail steel

L_{cont} = Length of structure between expansion joints

From Equation 2, it can be seen that the longitudinal rail stiffness of the unbroken rail is proportional to the area of the rail. The value is also proportional to the thermal force in the rail and the longitudinal

fastener restraint but these effects do not alter the rail stiffness significantly.

The longitudinal stiffness of the structure is found by analyzing the structure in a conventional manner by applying a longitudinal load and computing the deflection associated with this load.

The broken rail will slip until the tensile force in the rail is developed by the necessary number of fasteners. If the rail fasteners on one structure cannot develop the required tensile force in the rail, a portion of the pull apart force will be carried by adjacent structures, as shown in Figure IV-24b. The slip of the rail with respect to the deck of the guideway structure is determined by integrating the change in strain in the rail. This integration is represented by the area between the initial and final rail force curves divided by the AE of the rail (A is the cross-sectional area of the rail and E is the modulus of elasticity). The cross-hatched area in Figure IV-24b is the area representing the above integral. It is seen that the slip occurs on both sides of the pull-apart.

The unbroken rail stiffness can be determined by an iterative process. A force in the unbroken rail is assumed, the structure side sway necessary to develop this force is determined and the stiffness calculated. The distribution of the force can then be compared to the assumed force. The stiffness of the unbroken rail is not highly sensitive to the force in the rail, therefore, if the force in the unbroken rail is relatively close to the assumed force in this rail, the stiffness will not be significantly affected.

Once the force applied to the columns is determined, the side sway of the structure can be calculated. The pull-apart gap is then equal to the slip in the rail, as discussed above, plus twice the side sway of one structure. The factor of two on the side sway recognizes the condition of symmetry at the pull-apart.

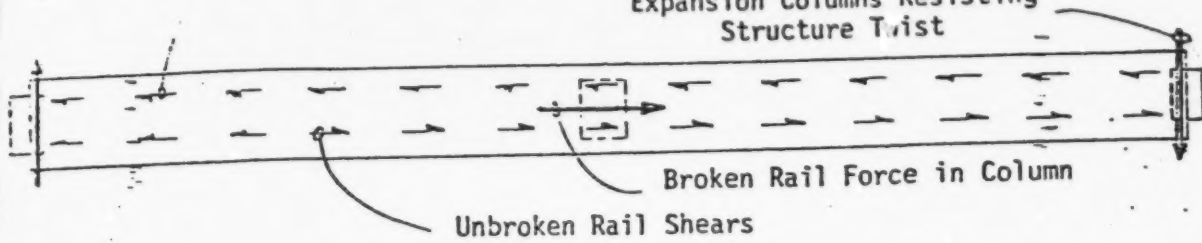
The shear between the rail fasteners and the deck apply a unit lateral bending moment to the structure. See Figure IV-25a. These moments are resisted by lateral shear in the beam structure and lateral forces at the top of the columns. The twist or rotation of the bridge structure is not significant. In essence the structure ignores the eccentricity and slides along its longitudinal axis.

The analysis of the broken rail for a dual-lane structure is similar to that previously discussed except that we now have three unbroken rails to share the force with the structure instead of only one unbroken rail. The stiffness of each unbroken rail is computed as stated previously, as is the pull-apart gap at the broken rail.

The twist of the structure is more cause for concern in a dual-lane structure since the eccentricity of the load with respect to the column axis is much larger for a dual-lane structure. An analysis of the guideway structure for the ICTS shows that the difference in forces in the unbroken rails due to column twisting is insignificant since again the twist of the structure is resisted by lateral shears in the beams. See Figure IV-25b.

Broken Rail Shears

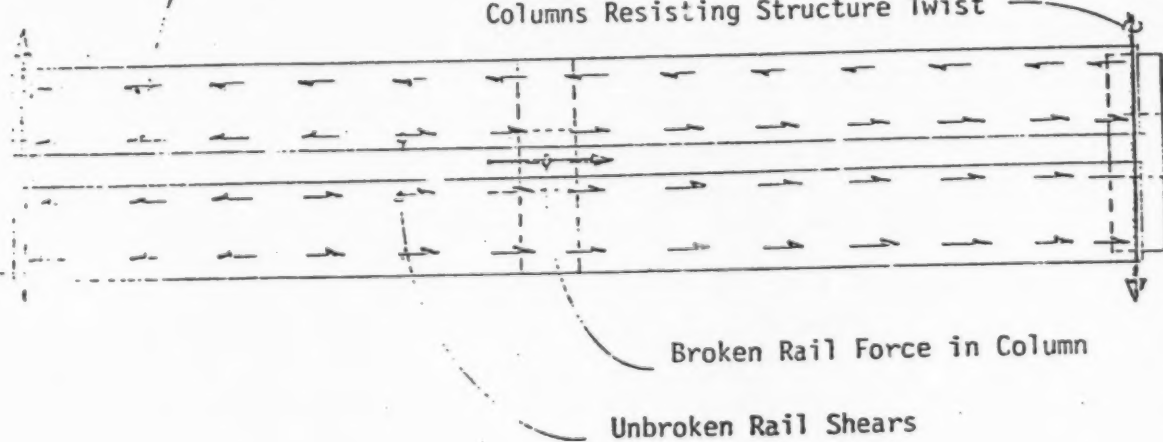
Lateral Forces at
Expansion Columns Resisting
Structure Twist



a. Single Lane Guideway

Broken Rail Shears

Lateral Forces at Expansion
Columns Resisting Structure Twist



b. Dual Lane Guideway

TRACKWORK STRUCTURE INTERACTION
BROKEN RAIL LOADINGS

FIGURE IV-25

EXAMPLE IV-4 BROKEN RAIL FORCE SINGLE LANE GUIDEWAY

Determine the distribution of the broken rail force in a single lane guideway.

Canadian Beam Cross Section.

Tangent 98.4 (30 m) spans.

Mixon Fasteners - 2400 lbs. longitudinal (10.7 kN)
restraint at 30 in o.c. (.762 m)

Column Height - 20 ft (6.1 m)

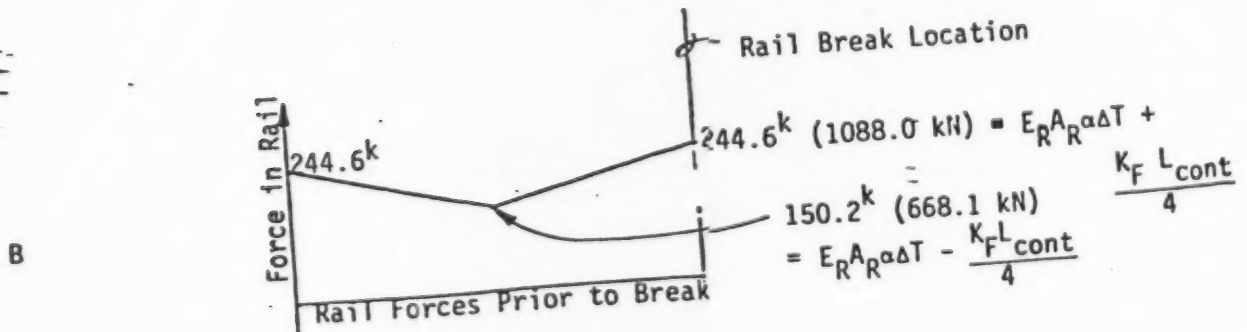
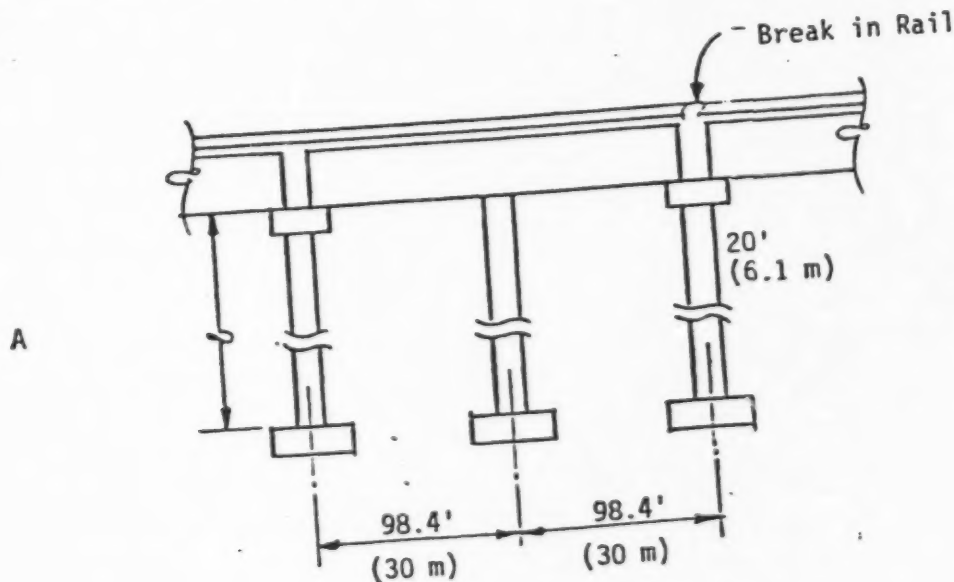
15 lb RE Rail Section -

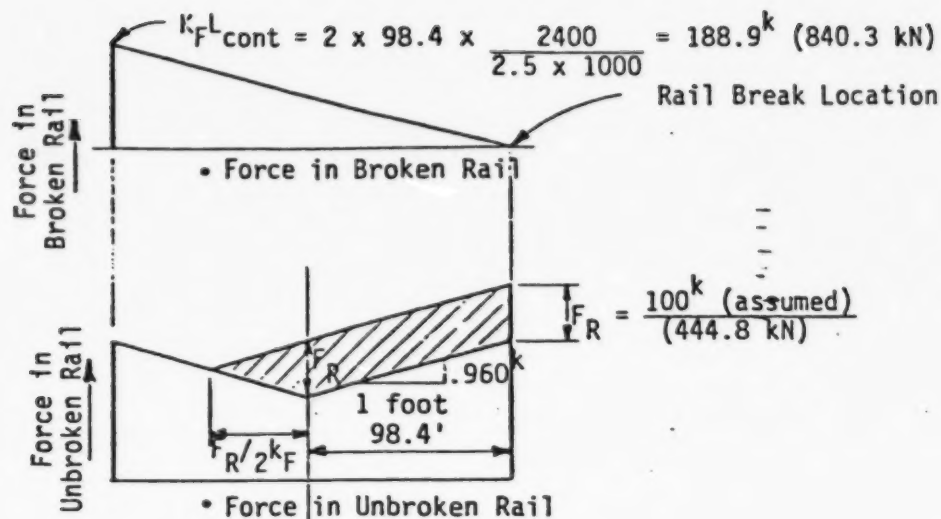
$$A_R = 11.25 \text{ in}^2 (72.6 \text{ cm}^2)$$

$$T = -40^\circ \text{ F. } (-40^\circ \text{ C})$$

$$E_R = 30,000 \text{ K/in}^2 (206,843 \text{ MPa})$$

$$\text{Design Stress Free Temperature} = +50^\circ \text{ F. } (+10^\circ \text{ C})$$





The shaded area in Figure D above = $A_R E_R \Delta_R$, where Δ_R = the elongation of the unbroken rail due to the force F_R .

$$A_R E_R \Delta_R = F_R \times 98.4 \times 12 + \frac{F_R^2 \times 12}{4 \times .960}$$

Assume: $F_R = 100^k (444.8 \text{ kN})$

Solving for Δ_R , using the above equation:

$$\Delta_R = .442 \text{ in.} \quad K_R = \frac{F_R}{\Delta_R} = \frac{100^k}{.442''} = 226 \text{ k/in } (39,578 \frac{\text{kN}}{\text{m}})$$

where: K_R = Longitudinal stiffness of the unbroken rail.

The structure longitudinal stiffness, K_S , from conventional sidesway analysis is:

$$K_S = 435 \text{ k/in } (76,180 \frac{\text{kN}}{\text{m}})$$

The 188.9^k force applied to the beam deck is the sum of the longitudinal fastener restraints from the location of the rail break. This buildup of force is shown in Figure 6 above. In long continuous structures the fasteners only must develop the rail thermal force plus the longitudinal restraint force. This force is distributed to the unbroken rail and the column in proportion to their longitudinal stiffnesses.

$$F_S = \text{Force Applied to Column} = \frac{K_S}{K_S + K_R} \times 188.9^k$$

$$= 124.3^k \text{ (552.9 kN)}$$

$$F_R = \text{Force Applied to Unbroken Rail} = \frac{K_R}{K_S + K_R} \times 188.9^k$$

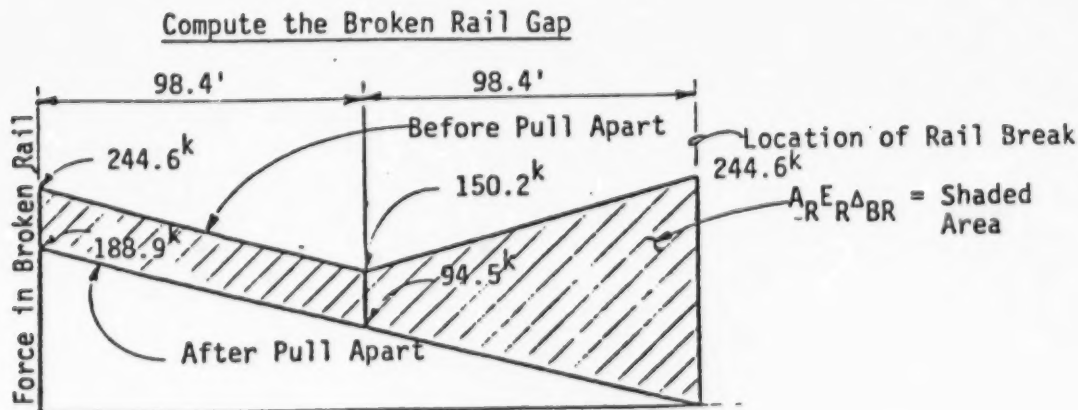
$$= 64.6^k \text{ (287.3 kN)}$$

Recomputing K_R using $F_R = 64.6^k$, $K_R = 244 \text{ k/in (42,730 } \frac{\text{kN}}{\text{m}})$

This stiffness results in the following distribution of the broken rail force:

$$F_S = 121.0^k \text{ (538.2 kN)}$$

$$F_R = 67.9^k \text{ (302.0 kN)} \cong 64.6^k \quad \text{Another iteration is not necessary.}$$



$$A_R E_R \Delta_{BR} = \left(\frac{244.6 + 55.7}{2} + 55.7 \right) \times 98.4 \times 12$$

Therefore:

$$\Delta_{BR} = .72 \text{ in (18 mm)} = \text{rail component of gap.}$$

$$\Delta_S = F_S/K_S = 121.0 \text{ k}/435 \text{ k/in} = .28 \text{ in (7 mm)} = \text{structure sidesway component of gap.}$$

Rail pull apart gap = $2(\Delta_R + \Delta_S) = 2.00 \text{ in (50 mm)}$, where the factor of 2 reflects the symmetry about the rail break location.

Compute Increase In Rail Stress-Unbroken Rail

Prior to Pull Apart:

$$\sigma_{\text{rail}} = \frac{244.6 \text{ k}}{11.25 \text{ in}^2} = 21.7 \text{ ksi (149.6 MPa)} \quad (\text{Figure B})$$

After Pull Apart:

$$\sigma_{\text{rail}} = \frac{244.6 \text{ k} + F_R}{A_R} = \frac{244.6 + 67.9}{11.25 \text{ in}^2} = 27.8 \text{ ksi (191.7 MPa)}$$

This stress is still well below the yield strength of rail steel.

EXAMPLE IV-5

BROKEN RAIL FORCE DUAL LANE STRUCTURE

Canadair Beam Section - Tangent 98.4 ft (30 m)

Hixson Fasteners = 2400 lb longitudinal
Restraint at 2.5 ft o.c. (0.762 m)

Col. Height = 23 ft (7.01 m) Canadair cross section)

115 lb RE Rail Section

Minimum T = -40° F. (-40° C)

Design Stress Free Temperature = $+50^{\circ}$ F. (10° C)

Force applied to beam deck by the broken rail = 188.9^k (840.3 kN), as in example IV-4 for single lane structures.

Assume: 1. 3 rails + structure share force.
2. Structure twist is not significant in computing rail stiffness.

Assume: $F_R = 35^k$ (155.7 kN)

The unbroken rail stiffness, K_R , is computed as in Example IV-4:

$$K_R = 262 \text{ k/in (458 kN/cm)}$$

$3K_R = 786 \text{ k/in (1376 kN/cm)}$, since the dual lane structure has
3 unbroken rails.

The longitudinal column stiffness, K_S , from conventional sidesway analysis is:

$$535 \text{ k/in (937 kN/cm)}$$

Therefore, the distribution of the broken rail force is:

$$F_S = \frac{K_S}{K_S + K_R} \times 188.9 = 76.5^k \text{ (340.3 kN) to the column, and}$$

$$F_R = 1/3 \times \frac{K_R}{K_S + K_R} \times 188.9 = 37.5^k \text{ (166.8 kN) to each unbroken rail.}$$

$\cong 35^k$ Another iteration is not necessary.

Compute Rail Gap

$\Delta_R = .72 \text{ in (18.3 mm)}$ as in Example IV-4.

$$\Delta_S = F_S / K_S = \frac{76.5^k}{535 \text{ k/in}} = 0.143 \text{ in (3.6 mm)}$$

$$\text{GAP} = 2(\Delta_R + \Delta_S) = 1.73 \text{ in (43.8 mm)}$$

d. Beam End Rotation:

At expansion joints, there is a change in slope between adjacent spans due to superimposed effects such as thermal gradients through the beam cross section and live load. This change in slope creates a bending stress in the rail in the vicinity of the joint.

The beam end rotation is dependent upon the stiffness of the girder and the length of the span. The curvature of the rail imposed by this change in slope is dependent upon the relative flexural stiffness of the rail and the stiffness of the resilient rail support pad. Figure IV-26 shows the effect of span length and rail section on the flexural stress in the rail. The curves were developed for a similar box beam section and a resilient pad stiffness of 180,000 pounds per inch (31.523 kN/mm). The analysis determines the curvature which stores the minimum strain energy in the system (rail and resilient pad). The analysis is relatively insensitive to pad stiffness.

The end rotation of the beam shall not be sufficient to cause fatigue problems in the rail.

Typical calculations for this effect are shown in Example IV-6.

EXAMPLE IV-6

RAIL STRESSES DUE TO LIVE LOAD END ROTATION OF BEAM

Structure : 2 Span Continuous - 98.4 ft (30 m) Span.

Beam Properties: $I = 601,530 \text{ in}^4$ (25,037,568 cm^4), CB-1 Cross Section
 $E = 4,000 \text{ ksi}$ (2,758 kN/cm^2)

Live Load : $W_{LL} = 1.33 \text{ kips/foot}$ (19.4 kN/m), including impact.

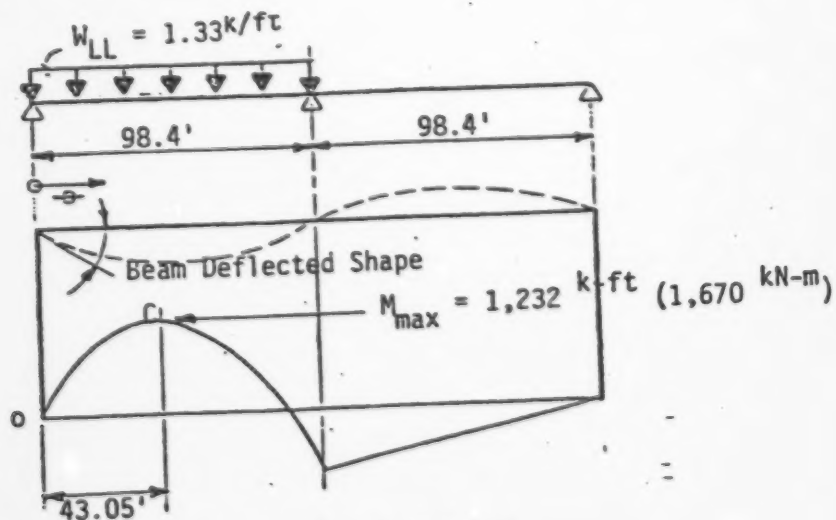
Rail Fastener Vertical Resilience : $K_V = 105 \text{ k/in}$ (183.9 kN/cm), Hixson Fastener.

Rail Properties: $A_R = 11.26 \text{ in}^2$ (72.6 cm^2), 115 lb AREA.
 $I_R = 64.34 \text{ in}^4$ (2,678.0 cm^4)
 $E_R = 30,000 \text{ ksi}$ (20,684 kN/cm^2)

ASSUMPTIONS:

1. Stiffening effect of rails not considered in beam deflection calculation.
2. The rail bends with a constant curvature (constant radius) at the expansion joint.
3. Rail fastener stiffness for uplift of rail is infinite.

BEAM END ROTATION:



Beam Bending Moment Diagram

The rotation of the free end of the beam is the integral of the beam curvature from the free end of the beam to the location of maximum curvature. Since curvature is related to bending moment, the beam end rotation is determined as follows:

$$\theta = \int_0^C \phi \, dx = \int_0^C \frac{M}{EI} \, dx$$

Where: M = Beam live load moment.
 E = Beam modulus of elasticity.
 I = Beam moment of inertia.
 ϕ = Curvature of the beam deflected shape.

Since both E and I are constant over the beam length, this integral equals the area under the moment diagram between the free end and point C divided by EI of the beam.

$$\theta = \frac{2(43.05 \text{ ft}) (1232 \text{ k-ft}) \times 144 \text{ in}^2/\text{ft}^2}{3 (4000 \text{ ksi}) (601,530 \text{ in}^4)}$$

$$\theta = \underline{\underline{0.00211 \text{ radians}}}$$

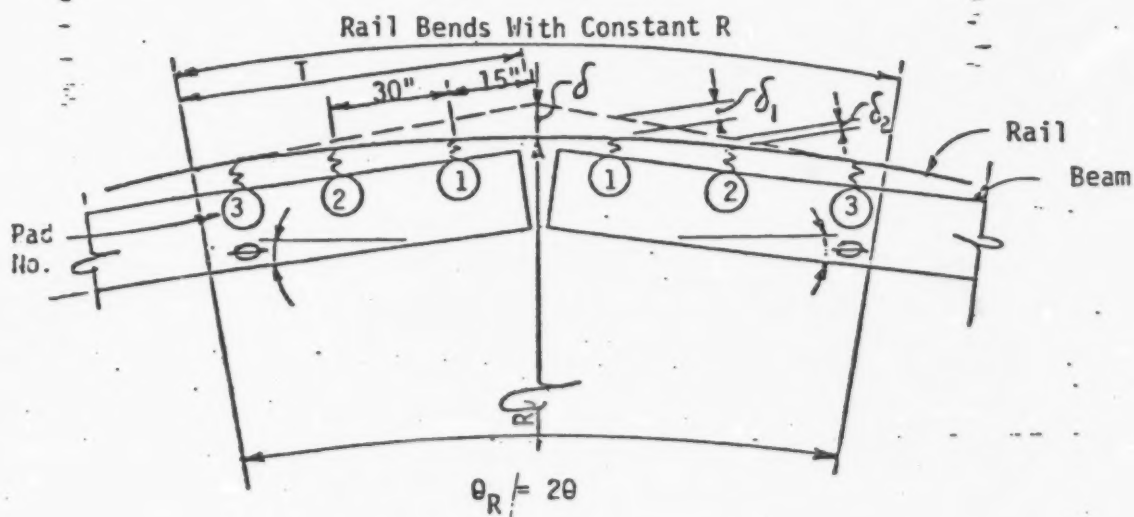
The strain energy due to flexure of the rail is:

$$U_{\text{rail}} = \int_L \frac{M_R^2}{2E_R I_R} \, ds, \text{ where}$$

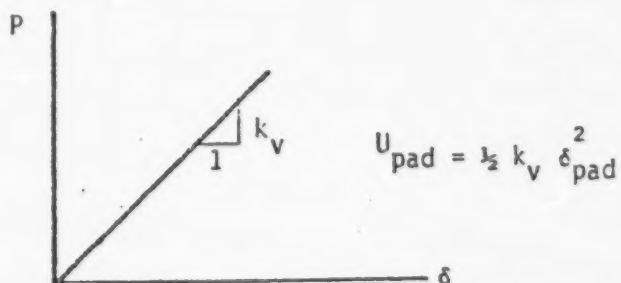
$$\frac{M_R}{E_R I_R} = \phi_R = \text{Curvature of the rail} = 1/R, \text{ therefore:}$$

$$U_{\text{rail}} = \int_L \frac{E_R I_R}{2R^2} (ds) = \frac{E_R I_R}{2R^2} (R\theta_R)$$

$$U_{\text{rail}} = \underline{\underline{\frac{E_R I_R \theta_R}{2R}}}$$



The strain energy for compression of the resilient pad of the fastener is:



$$\theta = 0.00211 \text{ radians}$$

$$\theta_R = 2\theta = .00422 \text{ radians} = 0.2418 \text{ degrees.}$$

Determine T, the length which gives minimum strain energy.

(a) Try T = 40":

$$R = \frac{T}{\tan \theta_R / 2} = 18,956 \text{ in (481.48 m)}$$

$$\delta = R \sec \left(\frac{\theta_R}{2} \right) - 1 = 0.0422 \text{ in (0.107 cm)}$$

$$\delta_1 = \frac{25''}{40''} (.0422'') = 0.0264'' (0.067 \text{ cm})$$

$$U_{\text{total}} = U_{\text{rail}} + U_{\text{pad}}$$

$$= \frac{E_R I_R \theta_R}{2R} + 1/2 K_V (\Sigma \delta^2_{\text{pad}})$$

$$= \frac{30,000 \text{ ksi} \times 64.45 \text{ in}^4 \times .00422 \text{ rad.}}{2 \times 18,956 \text{ in}} + 1/2 \times 105 \text{ k/in } (.0264 \text{ in})^2 \times 2$$

Due to Symmetry

$$= 0.215 \text{ k-in} + 0.073 \text{ k-in.}$$

$$U_{\text{total}} = 0.288 \text{ k-in (0.0325 kN-m)}$$

(b) Similar Calculations Yield:

<u>T</u>	<u>R</u>	<u>U_{rail}</u>	<u>U_{pad}</u>	<u>U_{total}</u>
45"	21,327 in. (541.71 m)	0.191 k-in (0.215 kN-m)	0.105 k-in (0.118 kN-m)	0.296 k-in (.0334 kN-m)
35"	16,588 in (421.33 m)	0.246 k-in (.0278 kN-m)	0.046 k-in (.0052 kN-m)	0.292 k-in (.0330 kN-m)

- (c) The conclusion is that the rail is bent to a radius of 18,956 in. in the condition which yields the minimum strain energy. This creates the following flexural stresses in the rail:

$$f_{\text{rail}} = \frac{M \cdot y}{I_R} = \frac{E_R I_R}{R} \frac{y}{I_R} = \frac{E_R y}{R}$$

Where: y = perpendicular distance from rail neutral axis to location where stress is calculated.

for 115 lb. AREA rail, $y_{\text{top}} = 3.47$ in (8.81 cm)

$y_{\text{bottom}} = 3.03$ in (7.70 cm)

$$F_{\text{top}} = \frac{30,000 \text{ ksi} \times 3.47 \text{ in.}}{18,956 \text{ in.}} = 5.49 \text{ ksi (tension)} \\ (3.79 \frac{\text{kN}}{\text{cm}^2})$$

$$f_{\text{bottom}} = \frac{30,000 \times 3.03}{18,956} = 4.80 \text{ ksi (compression)} \\ (3.31 \frac{\text{kN}}{\text{cm}^2})$$

These stresses are imposed on the rail every time a train crosses the structure. The tensile stresses in the head of the rail are additive to the thermal and longitudinal restraint rail stresses when rail temperatures are below the stress free temperature.

LIFTED FROM SAGS STRUCTURAL DESIGN CRITERIA

System performance criteria are excerpted from Guideway Design Guidelines for Hamilton Rapid Transit.

SYSTEM PERFORMANCE:

Performance Characteristics for a four-vehicle train

Max. Theoretical Sgl. Direction Cap.	25,400 pax/h
Max. Practical Sgl. Direction Cap.	19,900 pax/h
- Min. Theoretical Headway	- 47 sec
- Min. Practical Headway	- 60 sec
- Operating Modes	Scheduled
- Type of Vehicle Routing	Variable
- Traveling Unit	Up to 4 vehicle trains

VEHICLE:

Suspension Type	Steel wheels, 2 radial trucks per vehicle, 2 axles per truck as primary suspension; rolling sleeve airbag or rubber springs as secondary suspension.
Lateral Guidance	Coned, flanged steel wheels
Comfort	Enclosed vehicle, heated, air-conditioned and ventilated
Security	Two-way communication with central control
Overall Length (per mid-size car)	12,700 mm
Overall Width	2,500 mm
Overall Height	3,150 mm
Empty Weight	13,000 kg
Stopping Precision in Station	150 mm
One or Bi-Directional	Bi-Directional
Noise	66 dBA inside vehicle
Noise	67 dBA at 15 m from centerline of guideway
Total Pax Area (Centre Car)	24.9 m ²
Total Pax Area (End Car)	23.2 m ²
Floor Area for Seated Pax	12.0 m ²
Floor Area for Standing Pax	11.25 m ²
Vehicle Design Capacity	28 seated; 55 standing
Vehicle Crush Capacity	28 seated; 77 standing
Doorway Width	1,220 mm
Doorway Height	1,980 mm
No. of Doors per Vehicle	2 left side; 2 right side

PROPULSION:

2 linear induction motors (LIM) per vehicle	
Motor Rating	130 kw
Type Power	600 vdc
Power Collection	Truck-mounted side pick-up; two sets of shoes per truck
- Propulsion Input Power	Variable frequency and voltage
	3-phase AC
Vehicle Auxiliary System Input Power	37.5 VDC
Type Service Brakes	LIM in regenerative or dynamic mode. Disc brakes (pneumatic and hydraulic options) used for parking only in service mode.
Type Emergency Brakes	Electromagnetic track brakes assisted with disc brakes

SWITCHING:

Type and Emplacement	Track-based switching; movable frog or fixed frog
Switch Time (Lock-to-Lock)	3 seconds
Max. Speed Through Switch	- 72 km/h
Min. Headway Through Switch	60 seconds

STATIONS:

Type	On-line or Off-line
Type Boarding	Level
Minimum Vehicle Station Dwell Time	5 sec

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.3

STRUCTURAL AND AESTHETIC CONSIDERATIONS

COLE SHERMAN & ASSOCIATES LIMITED,
TORONTO, ONTARIO

JUNE 1981

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

INDEX

	<u>PAGE</u>
SUMMARY	1
1. INTRODUCTION	3
2. TRAPEZOIDAL PROPORTIONS	6
3. DESIGN PRIORITIES	8
4. DETAILED DESIGNS	10
5. TOLERANCES	19
6. CORROSION DUE TO STRAY ELECTRIC CURRENTS	21
7. AESTHETICS	23
8. CONCRETE - STEEL COMPARISON	28
9. FUTURE CONSIDERATIONS & RECOMMENDATIONS	35
10. ITEMS EXCLUDED FROM THE STUDY	37
11. REFERENCES	39

LIST OF TABLES AND FIGURES

TABLE 1.	Typical % Cost Distribution - Concrete
TABLE 2.	Typical % Cost Distribution - Steel
TABLE 3.	Summary of Box Girder Properties
TABLE 4.	Summary of Box Girder Properties
TABLE 5.	Unit Prices - Concrete
TABLE 6.	Unit Prices - Steel
TABLE 7.	Alignment Costs - Steel and Concrete
TABLE 8.	Comparison of Unit Configuration Costs - Concrete and Steel
TABLE 9.	Comparison of Total Project Cost of Steel and Concrete Alignments
FIGURE 1.	M.T.C. Box Girder Curve
FIGURE 2.	Perspective View of Steel Guideway
FIGURE 3.	Composite Steel - Concrete Girder at Expansion Support
FIGURE 4.	Composite Steel - Concrete Girder at Fixed Support
FIGURE 5.	Steel Guideway - Conceptual Layout
FIGURE 6.	Steel Guideway - Expansion Joint Details
FIGURE 7.	Steel Guideway - Pre-Cast Deck Details
FIGURE 8a.	Vertical Alignment Control - Steel Guideway
FIGURE 8b.	Vertical Alignment Control - Concrete Guideway
FIGURE 9.	Weight of Steel V/S Span
FIGURE 10.	Construction Schedule
FIGURE 11.	Construction Sequence - Steel
FIGURE 12.	Construction Sequence - Concrete

SUMMARY

This study has examined the viability of steel as an alternative guideway material to concrete from the viewpoint of structural adequacy and architectural appearance, the latter with the assistance of T. Garwood-Jones, Architect.

By agreement, only a trapezoidal composite concrete/steel box girder has been examined in a two span (30 m) continuous configuration. This box girder reflects the shape of the concrete box girder already developed. The foundation loads are almost identical to those generated by the concrete guideway and it has been assumed that the choice of guideway material will not affect foundation feasibility or cost.

Design of the steel box girder and column-crosshead meets the major requirements of the Design Criteria prepared by ABAM Limited for the study utilizing analyses aided by the MTC computer programs. The basic comfort criteria required for the ICTS system have been met. Allowance has been made in the dead loads for possible sound damping materials.

Greater emphasis has been placed in the study on those guideway elements in which steel appeared to be the most attractive as an alternate guideway material. Thus, more effort has been expended in examining tangent sections and large radius bends, than on any other elements, although the examination of the other elements in steel has been taken far enough to establish viability.

The implications of incorporating conduits, a deck drainage system and permanent or temporary bracing have been considered in the designs and vertical construction tolerances have been compared for both concrete and steel. A cursory examination of the possible effects of stray DC current has been included.

The architectural impact of the guideway using weathering steel as the girder material has been considered together with the associated precautions to be taken.

Comparisons have been made between the concrete guideway and steel alternative on the basis of cost, supply and delivery, and constructability.

Constructibility has been investigated with particular reference to construction time and urban impact resulting from construction.

A total guideway cost has been calculated for each material based on a hypothetical but representative Hamilton alignment.

While further study is recommended on several items, some of the main study conclusions are:

1. The composite steel/concrete trapezoidal box section girder is clearly a viable alternative to a concrete box girder in the Hamilton ICTS guideway system.
2. The strength, cost, feasibility, aesthetic, ride comfort and noise radiation standards set by the ICT system can be met by the steel alternative guideway system to the same degree as the concrete guideway system.
3. Weathering steel, as an architectural material, offers a distinctly different appearance to that of concrete, particularly at close range. However, colour may have only a marginal visual impact as compared with the impact of the guideway mass as a whole. The slightly reduced beam depth and smaller circular section columns of the composite guideway offer some reduction in visual impact over the concrete alternative.

1. INTRODUCTION

Since the Provincial Government's announcement in 1972 of a \$1.3 billion rapid transit program, an intermittent dialogue has taken place on the merits of steel as a guideway material. Both the steel industry and the Government agencies associated with transit have failed to draw conclusions as to the role of steel in the construction of an elevated transit system.

During this time,, a number of U.S. cities have acquired elevated steel guideway systems and a considerable number of steel and composite concrete elevated road structures of similar span length have been constructed in Ontario including Hamilton.

Development of the ICTS guideway system by UTDC/MCL has proceeded rapidly assuming a concrete superstructure. During this process, certain problems have been identified in such areas as feasibility, cost, design criteria, etc; all of which have been solved to satisfy the needs of the system. However it seems that at each stage of development in the concrete system, the solving of a particular problem has given rise to the fear that a corresponding solution may not exist for a steel alternative and possibly there might be other, as yet, unidentified "steel" problems without solutions.

This Study, by Cole, Sherman & Associates Limited, was initiated by the Region of Hamilton-Wentworth through MCL to briefly examine some of these concerns and to assess the viability of a steel guideway alternative for the Hamilton area.

The initial phase of the Study was undertaken by MCL and concluded that a trapezoidal box section was considered to be the most appropriate steel shape to be examined.

The role of Cole, Sherman & Associates Limited in this second phase of the Study, is expressed in the enclosed structural work statement and considers cost, feasibility and constructibility of a steel guideway structure consisting of a steel trapezoidal box with a composite concrete deck. Also discussed in the Study are the concerns for aesthetics, effects of sound damping and the use of weathering steel. This forms only part of the overall examination carried out by MCL which

includes vehicle guideway interaction, environmental impact and sound transmission.

Time and other constraints necessitated the adoption of certain assumptions and conditions for this Study. Any further pursuit of a steel guideway development should begin with a re-examination of these assumptions.

It has been assumed in the Study that a typical guideway consists of consecutive two span continuous beams of trapezoidal shape. A typical guideway alignment comprising representative lengths of single and dual tangent section and 35 m radius curves has been adopted to provide a comparison between the steel and concrete superstructures. Except for a few minor instances, guideway loadings have been taken directly from those used in the concrete guideway design. A 30 m span has been assumed for all tangent sections of guideway and an 18 m span for the sharp radius bends. The Study concentrates on the more common guideway elements rather than the more complicated and less frequent items. This Study is consequently not exhaustive although the intention has been to make optimum use of in the limited time allowed for investigation.

STRUCTURAL WORK STATEMENT

Structural and Aesthetics - Cole, Sherman & Associates

1. Study objective is to establish whether steel is a viable alternate to concrete for a guideway structure.
2. Investigate the trapezoid proportions discussed in the MTC paper dated April 1979. Determine the most suitable section for a trapezoidal shape steel guideway straight beam applicable to a span of 100 feet and to the maximum number of special beam configurations.
3. Develop preliminary design concepts for a variety of span configurations, using candidate trapezoidal steel section (straight, horizontal and vertical simple curves, cross heads, columns, spiral curves, crossovers and other special configurations). Prioritize on the basis of frequency constructibility and designability.
4. Develop detailed designs in priority sequence.
5. Review aesthetics, particularly weathering (with C.I.S.C. assistance) and noise and vibration control systems with noise and vibration consultant.
6. Compare candidate trapezoidal steel section with trapezoidal concrete section for cost, supply and delivery, constructability.
7. Prepare cost comparison of steel vs. concrete span configurations for the relevant parts of a typical alignment.
8. Prepare report on structural and aesthetics as an appendix to the feasibility report.

2. TRAPEZOIDAL PROPORTIONS

- 2.1 Phase I of this Study, by MCL, consisting essentially of a review of relevant guideway literature concluded that the trapezoidal box section was considered to be the most appropriate for high quality urban integration. This part of the Study, Phase II, has proceeded on this basic premise.
- 2.2 In view of the time constraints imposed on the Study of a Steel Alternative, it seemed reasonable to assume that the general relationships that exist between concrete and steel structures of this type will apply to the Hamilton guideway. In particular it is usually found that optimum steel beam alternatives are generally shallower than comparable concrete beams. Consequently a steel box section 1,150 mm (45.3") deep from top of steel flange to beam underside was examined as a possible contender. This is a 4% reduction in overall depth from the concrete section. The MTC report entitled "An Evaluation of Steel and Concrete Guideway Sections for I.C.T.S." dated April 1979 contains considerable data on the sizing of trapezoidal box sections which is summarized in the Figure 1 for simple spans. Modification of this Figure for the reduced live load deflection following from the use of a two-span continuous beam indicated that a 1,150 mm (45.3") deep beam would satisfy the deflection limitation of span/800 for a 30 m (100') span with enough latitude to enable the designers to stretch the span in special cases.
- 2.3 As the two-span configuration reduces bending moment in the midspan box section (as compared with that in the simple span developed in the MTC Study) and hence the thickness of the bottom flange required for strength), it was considered prudent at the preliminary design stage to reduce the box beam width to less than that suggested by Figure 1. Rail fastener-shear connector interference could best be controlled under this condition if the centreline of the box beam web coincided with the rail centreline. Moving the top flange of the box girder to a position inside the rail fixing device would introduce the likelihood of design problems with the concrete top slab/parapet wall which could not be considered in this Study and would result in a steel box girder of unusually narrow proportions. This could be a disadvantage in the curved beam sections where maximum torsional stiffness would be desirable.

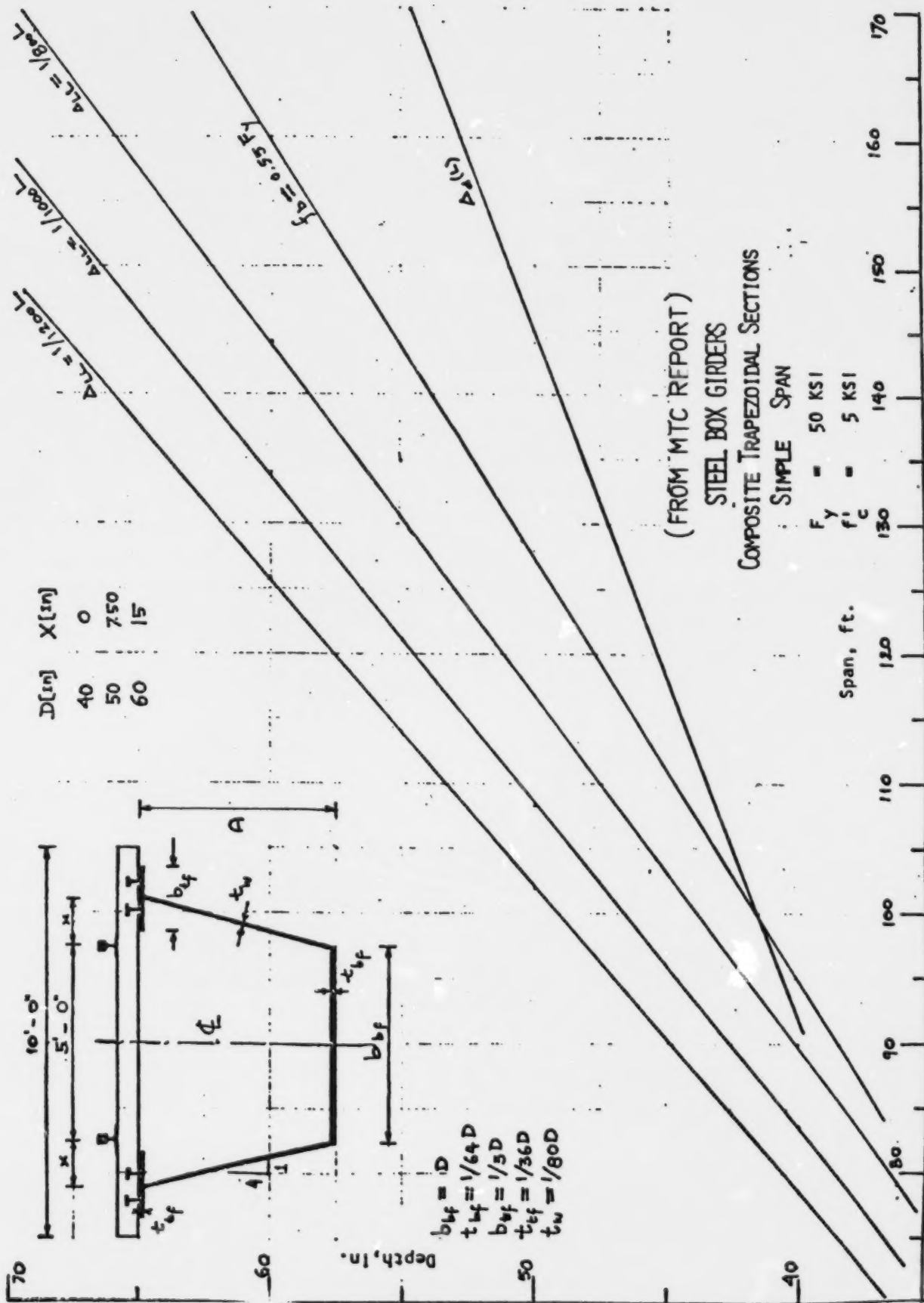


Figure 1

- 2.4 The 4 to 1 slope of the box girder web was maintained, thereby permitting the minimum width bottom flange. This is generally accepted as being the greatest permissible slope. Preliminary discussions with one steel fabricator through the co-operation of C.I.S.C. confirmed that Grade 350 MPa (50.8 ksi) structural steel should be considered in the preliminary designs as it is readily available in both weathering and high toughness material.
- 2.5 The candidate section properties are given in Tables 3 and 4 and the proportions in Figures 3 to 7.

3. DESIGN PRIORITIES

- 3.1 Table 1 illustrates in general terms the components of a typical Hamilton concrete guideway alignment and lists their percentage contribution to the total guideway cost. These were calculated from average unit costs developed for the concrete guideway study. Associated with each element of the guideway is a description of the measure of design or construction difficulty anticipated. These percentages and consideration of design difficulty and constructibility have been used to establish the order in which each configuration was examined in the Study and the depth of this examination. Those configurations with highest cost contribution and most straightforward design input, dual tangent and single tangent, have been considered in more detail. In cases where cost contribution is small more emphasis has been placed on constructibility rather than on actual cost, particularly where design is complicated. For instance in Table 2 the contribution to cost of the column/crosshead for dual guideway tangent section is approximately double that for single plus dual 35 m curved section guideway beam. This places it well ahead of curved guideway section beam in order of priority due to its less complicated design input.

For this Study constructibility has to be considered in terms of average construction conditions. The precise assessment of constructibility is generally associated with extensive design input requiring the expense of manpower beyond the scope of this Study.

- 3.2 Applying any kind of value system to the three factors, total cost, constructibility and designability would not affect placing the dual tangent guideway section at the position of first priority in this Study, together with the single tangent which includes similar and, in general, simpler details. Column and crosshead feasibility clearly contributes an important factor to the Study and was placed next in order but with some limitation on the depth of analysis and number of column heights considered.
- 3.3 The 35 m curve forms a fairly insignificant proportion of total construction cost and is complicated to analyze. Investigation into sharp radius curves, of necessity, has only been pursued at a very preliminary level to establish the actual order of magnitude costs and possible construction advantages of steel over concrete in this area.

Configuration	Beam	Column-Crosshead	Foundation	Total	Constructibility	Designability
Single Tangent	8	1	3	12	Straight forward	Simple
Single 35 m Curve	2	1	2	4	Difficult	Complicated
Dual Tangent	28	3	6	37	Straight forward	Routine
Dual 35 m Curve	2	1	1	2	Difficult	Max. complicated
Dual 300 to 35 m Curve	6	1	1	8	Straight forward to difficult	Routine
Tunnel Section	37	N.A.	N.A.	37	N.A.	N.A.
TOTALS	83	5	12	100%		

TYPICAL GUIDEWAY % COST DISTRIBUTION - CONCRETE

TABLE 1

Developed from final costs (Table 7) as a Comparison with Table 2 - Steel

Priority Order:

1. Single and Dual Tangent
2. Column - Crosshead Feasibility
3. 35 m Curve Guideway
4. Large Radius Curve.

Configuration	Beam	Column	Foundation	Total	Constructibility	Designability
Single Tangent	7	1	3	11	Straight forward	Simple
Single 35 m Curve	1	<1	2	3	Difficult	Complicated
Dual Tangent	29	4	6	38	Straight forward	Routine
Dual 35 m Curve	1	<1	1	2	Difficult	Max. complicated
Dual 300 to 35 m Curve	5	1	1	7	Straight forward to difficult	Routine
Tunnel Section	38	N.A.	N.A.	38	N.A.	N.A.
TOTAL	81	6	13	100%		

TYPICAL GUIDEWAY % COST DISTRIBUTION - STEEL

TABLE 2

Developed from final costs (Table 7) as a comparison with Table 1 - Concrete

Only one configuration consisting of 18 m (59') spans on 90° turns with 35 m radius was considered and this was limited to the box section itself. It has been assumed that the sizes of columns, crossheads and foundations are in proportion to the relevant steel tangent or concrete curved beam solution.

- 3.4 In contrast the significance of tangent section warrants an investigation of viability in a range of span possibilities. This Study discusses spans of 24 m (79'), 30 m (98.5') and 36 m (118') for tangent section.
- 3.5 Preliminary discussions with the steel industry have confirmed that very large radius horizontal and vertical curves do not present any additional fabrication problems compared with tangent section in general. Spiral approaches to sharp radius curves would, however, present similar difficulties and increases in cost to those of the sharp radius curves themselves.

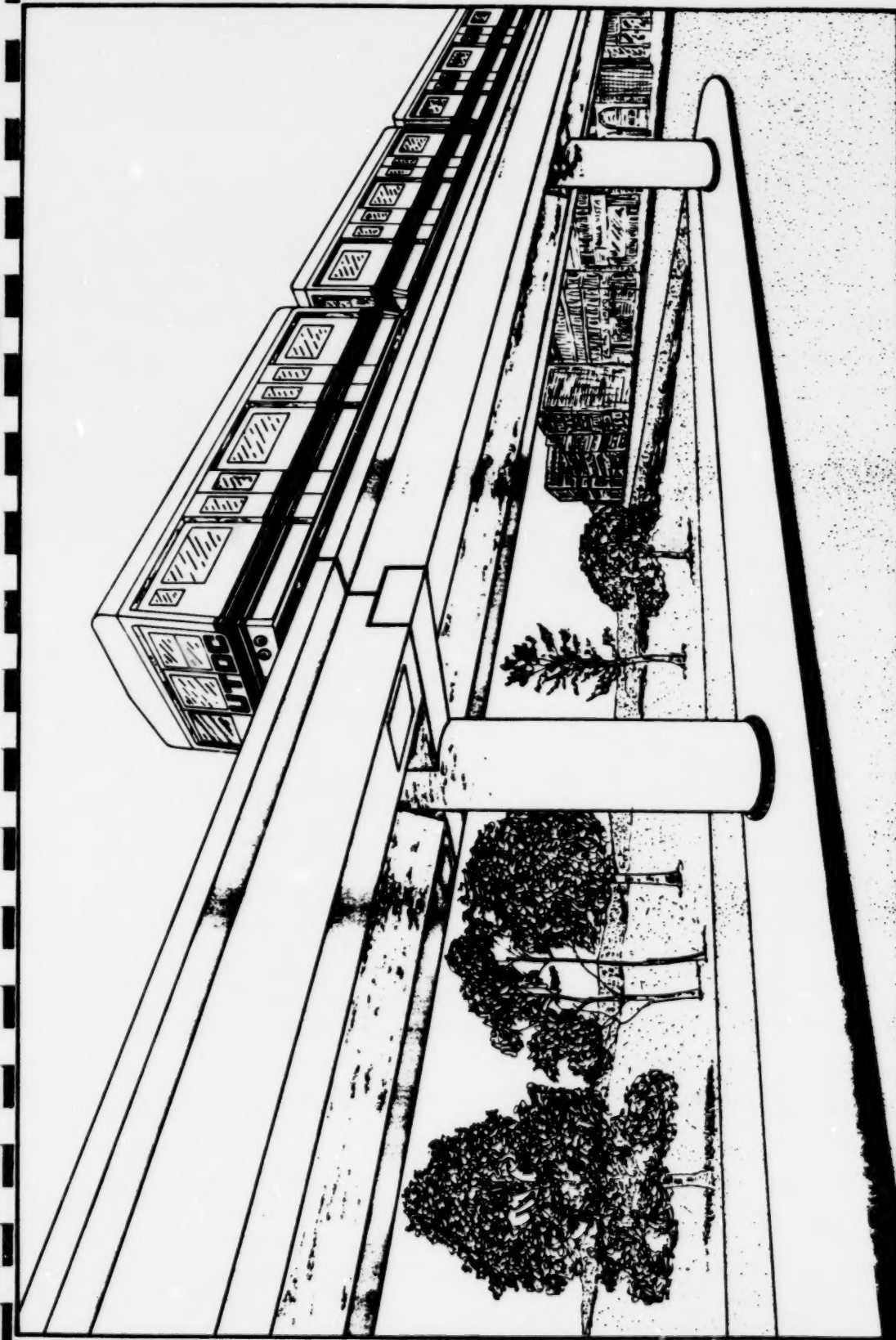


FIGURE 2
PERSPECTIVE VIEW OF STEEL GUIDEWAY

CS-834

4. DETAILED DESIGNS

4.1 Design Criteria

The Preliminary Design Criteria for a Guideway Structure dated February 25, 1981 and prepared by ABAM Limited as a draft, forms the basis for detailed designs of the steel guideway alternative and is appended to this Study.

With the co-operation of The Ministry of Transportation and Communications computer, an analysis of beam sections has been carried out using the Continuous Steel Girder Program BROO260 and the Continuous Curved Bridge Program BR00210. Both programs use AASHTO (Standard Specification for Highway Bridges - The American Association of State Highway and Transportation Officials) as a basis for design parameters such as fatigue and width thickness ratios. In addition they will accept any material properties and load configuration desired by the user.

However, load factors used in program based on AASHTO analysis, have been adjusted to suit guideway design criteria. The width thickness ratios for the top and bottom flanges and webs have been proportioned to satisfy the AASHTO box girder requirements. The AASHTO fatigue requirement has been satisfied for over 2 million cycles for critical loads. This is the upper limit for which highway and railway bridges have been designed in North America.

The design of the guideway for 5.5 million cycles for the respective critical loads, more detailed analysis, has yet to be verified and employed in a future study.

Reference, where necessary, has also been made to the Ontario Highway Bridge Design Code, which is an extensive source of directives for bridge design and also the most up-to-date document of its kind.

4.2 Limitations

Investigating the viability of a steel alternative in the short period of time available called for some considerable economy of effort on the part of Cole,

Sherman & Associates Limited. Because of the time frame, there was obviously no way in which every design requirement of the design criteria could be applied to each part of the candidate section and its supports.

There were several boundaries which could be applied to the investigation and these were used to place reasonable limits on the development detail of a constituent part. These boundaries, expressed in the form of questions, are as follows:

- (i) Can parts of the concrete guideway design be applied to the design of the steel alternative and obviate the need for any further investigation in this Study?
- (ii) Can the constituent part of the guideway under review be constructed in steel?
- (iii) Are the parameters affecting design of the steel guideway items the same or similar to those for the concrete guideway?
- (iv) As a test of viability, is the cost of the steel guideway item within a reasonable percentage difference of the comparable concrete item? Say 10%.

The following answers defined the limits of the investigation and established its validity:

- 4.2.1 In answer to (i), it has been assumed that the concrete top flange and sidewalls of the concrete box girder form the lid of the steel box girder and that minor modifications to reinforcing or underside shape to satisfy the needs of a slightly longer deck cantilever can be provided at negligible cost.

Although the steel guideway deadload is less than the concrete guideway section and therefore enabling a possible reduction in footing size, the cost per unit length of guideway for foundations has been retained. A variation of 10% in foundation cost would represent only 1.3% of the total project cost. It has been recognized, however, that in cases of high lateral loading such as sharp curves, a reduced deadload could increase the cost of foundations above that for the concrete guideway.

The steel box section along with supporting columns and crosshead have been investigated. A construction cost has been obtained for the concrete deck using the concrete guideway design shape in order to complete costing data. The cost of a precast alternative has also been obtained.

- 4.2.2 Can the constituent part under review be constructed in steel to satisfy the needs of the ICTS System in Hamilton? The priority value indicated by Table 1 and discussed previously has been used to establish the feasibility only of using steel for those items which form a small part of the total construction cost. For column/crossheads the question of feasibility can be quickly answered by the application of the general principles of fabrication methods and then the sizes of steel plates determined by analysis using the dominant force systems. An overestimate in design of column/crossheads in the order of about 25% only influences the total cost of the project by about 1.5%. For this reason, only one span length and column height has been considered to provide adequate data on which to determine viability of the steel alternative column/crosshead. However, in the case of the box girder itself including the concrete lid, forming about 43% of the project construction cost (see Table 2), more extensive investigation has been carried with an accuracy of about 10% in its cost. This possible variation represents about 4.3% of the total project cost.

Three box girder spans have been analyzed in detail with spans of 24 m (79'), 30 m (98.5') and 36 m (118') to indicate whether there is any advantage in increasing or decreasing the average span of about 30 m in the concrete guideway. Construction methods, splice points, etc., have also to be identified for the box sections as well as the thicknesses and curtailment lengths of plates. The requirement for any bracings, temporary or permanent has also been established. Erection methods and scheduling of fabrication and erection has also been addressed.

- 4.2.3 All parameters affecting design of the steel guideway alternative, question (iii), have been assumed the same as those for the concrete guideway except as follows:

Gravity Loads

Gravity loads are less for the steel box section than for the concrete but all other superimposed deadloads such as LIM rail, etc. are the same.

Sound Damping

There seems to be some evidence available that noise damping may be required for steel box girders of this size, and the analysis has assumed a 10% increase of gross deadload as an allowance for sound damping materials. This will be required in present and future residential areas of the city. The steel guideway estimate has been prepared with the sound damping for the full length of typical alignment, considering possible future residential developments.

Impact

The preliminary design criteria states that an impact factor of 10% - that used for the concrete guideway - does not apply to the steel alternative and that a higher value should be assumed.

Impact effect according to the reference material is generated by out-of-balance vehicle mechanisms, track roughness, and vehicle/guideway interaction. The first two sources of impact should be similar for both alternatives. Vehicle/guideway interaction is a function of vehicle mass and speed, and stiffness and mass of the supporting structure. The design criteria attempt to control rider comfort by placing limits on the value of beam deflection in relation to span and on the value of the first mode of natural frequency of vibration of the girders. Whichever of these two limits applies there is a high probability that the steel guideway structure will be lighter in weight than the equivalent concrete guideway structure, consequently its dynamic response to vehicle loading will be greater. In the absence of sufficient data to quantify this effect, an impact factor of 15% has been assumed for the steel guideway alternative. This increase is also supported by the fact that in general the percentage of critical damping for steel structures tends to be less than that for concrete. This increases the probability that the guideway would still be vibrating from the passage of one vehicle as a second begins to cross, adding marginally to the maximum amplitude of vibration of the guideway and the effect of impact.

Ultimate Strength

The side walls have not been included in derivation of guideway beam strength. This is partly because the computer analysis program does not readily receive a

concrete box lid of that shape and partly because past experience dictates that caution should be observed in relying on these appendages for strength. From a strictly practical viewpoint, it is not easy to see how full ultimate strength could be developed in the gross section due to shear lag, lateral instability and lack of containment of the narrow wall. Full section has, however, been used in calculating section properties for frequency of vibration and live load deflection of the girder.

All remaining design criteria have been assumed the same for both alternatives.

- 4.2.4 In answer to (iv), steel guideway costs have been developed from design sketches prepared by Cole, Sherman & Associates Limited through the co-operation of C.I.S.C. (Canadian Institute of Steel Construction).

These costs have been added to the estimated cost of constructing the concrete deck and to the assumed foundation cost taken from the concrete guideway study.

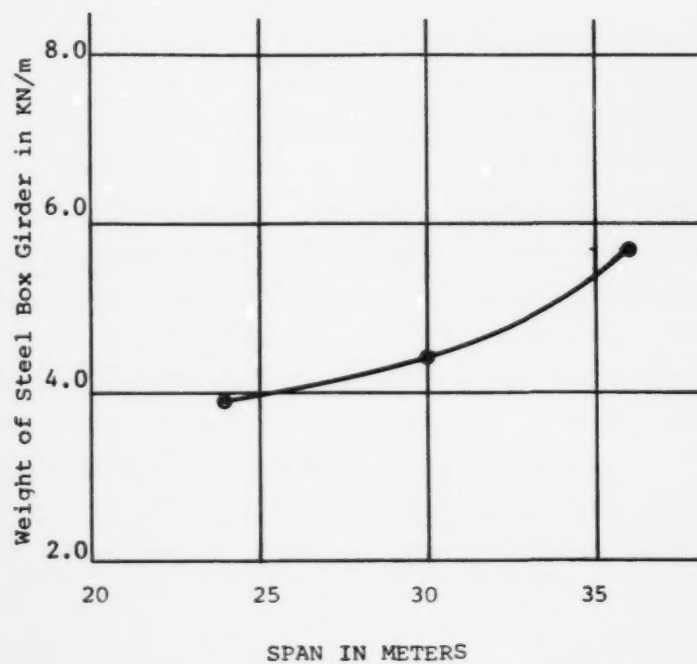
Unit prices include subcontractors overhead and profit plus a contingency allowance, and are current for January 1981 as are the concrete guideway figures.

See Cost Comparison Section for comment.

4.3 Designs

4.3.1 Box Girder Design - Tangent Section

Shortage of time did not permit investigation of all the possible combinations of loading described as U_1 through U_8 in the Design Criteria, therefore the box girder has been designed to satisfy the few combinations which were judged most critical. Experience suggests that the most severe system of loads influencing girder design are the primary load system of gravity and vehicle loads, and for steel, the frequency of load repetition. Controls are also placed on frequency of vibration of the girder - first mode and on its deflection to span ratio. Load combination U_1 gives greater emphasis to the primary load system and this has been applied to the beam for ultimate strength calculation.



WEIGHT OF STEEL BEAM V/S SPAN

Figure 9

The histogram of vehicle loads given in the design criteria shows that the probable maximum live load - the crush load - will occur about 140,000 times during the guideway lifetime and that a 2/3 full vehicle live load will occur about 1.9 million times. This latter frequency has greatest influence on the fatigue requirements of the design. These two loads have been considered in the analysis. The load cases consisting of the highly improbable and infrequent crush load with snow and ice and the half full vehicle with 5.5 million cycles of repetition have not been considered because of the unavailability of a suitable analysis program.

The computer analysis program used contains the facility of being able to run a ten axle vehicle across the two-span structure. By combining the vehicles double axles into one load, it has been possible to create the equivalent of a five vehicle consist, although only 4 vehicle consists are presently envisaged for Hamilton. The difference in effect is approximately 5% greater than that for a four vehicle consists. Values of the axle loads have been modified to correct the program's built-in AASHTO load factor to match the value of 'two' given in the design criteria. As well as checking ultimate load conditions, the beam computer program also outputs service load stresses for comparison with maximum allowable fatigue stress and modifies shear connector spacing accordingly.

Web and flange width to thickness ratios, and stiffener requirements for the box beams, are in accordance with AASHTO specifications, and were checked manually as were the stiffness properties for calculating natural frequency and live load deflections of the guideway.

Secondary effects due to eccentric lateral loads on the box section were also examined manually to confirm their second order values.

Derivation of final box girder sizes is an iterative process consisting of cycles of activity beginning with an estimated size, going through computer analysis and then manually checking other items such as frequency of vibration and reviewing constructibility with the steel industry.

Several cycles of this process were performed on the three spans length of box beam girders to arrive at the design recorded on the enclosed sketches. Figure 9 gives the final weights of steel in the box girders for each span and it can be seen that the weight increases progressively with span, but at the lower range is becoming almost constant as minimum plate thicknesses are being used.

Natural frequency of girder vibration has been calculated assuming full fixity of the girder at the fixed centre column with full participation of the concrete sidewalls. This assumption leads to higher values of frequency (50% higher) than occur in the case of simply supported spans.

4.3.2 Column-Crosshead-Tangent Section-Dual Guideway

Only two sizes of crosshead have been investigated. One at the typical expansion joint and one at the typical fixed support. Both crossheads conform to the requirement that their undersides and ends be flush with the box beams and that they provide a bearing seat geometrically similar to that of precast concrete at the expansion joint. It is the intention of this design that interconnection will be possible between steel girders and concrete crossheads and between concrete girders and steel crossheads.

Torsional strength requirements naturally lead to the choice of a box section crosshead beam for greater economy, especially as this provides a similar bearing arrangement at the expansion end to that of the concrete guideway, which could facilitate interconnection between steel and concrete guideways.

Torsional moments are greatest in the crosshead when full live load is applied to one span assuming full column fixity. Bending moments and shears are maximum for two continuous live load spans and these two conditions have been considered in the design.

Economy demanded the choice of circular steel columns as opposed to rectangular fabricated shapes. Columns are concrete filled and designed to act compositely. The primary force system most critical for column design was not immediately apparent. Various combinations of vertical loads, either central or eccentric are possible in combination with a series of lateral and/or longitudinal

forces. In this Study for tangent section columns, it has been found from approximate calculations that the load combinations containing wind load, earthquake or accidental vehicle collision have a similar effect, but the one containing maximum eccentric live load produces the worst effect and this was the one that was applied.

Investigation into the ductility of the column base connection for earthquake forces was not considered.

4.3.3 Column - Single Guideway

In order to establish an order of magnitude size for single guideway columns, the design of a 6 m high column has been carried out using similar assumptions to those of the dual guideway column.

4.3.4 35 m Radius Curve

Preliminary estimates of the box girder size suggested that the section analyzed for the 24 m span of tangent section would prove to be adequate for an 18 m curved span of 35 m radius. This design was subsequently analyzed for the same load cases used on the typical tangent sections, combining the curved beam computer program output for torsional effects with longitudinal bending moments derived from the beam program. Top flange diagonal bracing was required to provide the open box with torsional continuity during construction of the concrete deck and control deflections. With all effects combined the chosen section was found to be more than adequate, but as its webs and flanges are already minimum thickness for avoiding stiffeners, which add to the cost, no further size reduction was attempted.

The size of column required to resist the horizontal continuous rail force on 35 m radius bends has been determined by preliminary calculation but the crosshead has been assumed to be similar to that for dual tangent section.

4.3.5 Applied Design Criteria - Summary

Please refer to Preliminary Design Criteria prepared by ABAM Limited for symbols.

(1) Box Girder - Tangent Section

- . Crush load vehicle with deadload and 10^5 cycles
- . 2/3 full vehicle with deadload and more than 2×10^6 cycles.
- . Ultimate Strength Load Combination U_1 with both above vehicles.
- . Service load condition for fatigue.
- . Natural frequency not less than 3 Hz.

Analysis for 24 m, 30 m and 36 m spans using these criteria.

(2) Crosshead - Column - Dual Guideway

- . Crush load vehicle to give maximum eccentricity with deadload, hunting and emergency braking.
- . Ultimate strength Load Combination U_1 .
- . Two conditions - maximum bending, maximum torsion.

(3) Box Girder - 35 m Radius Section

- . Same as Tangent Section for longitudinal effects.
- . Calculation of torsional effects due to curve.

Analysis for 18 m span only.

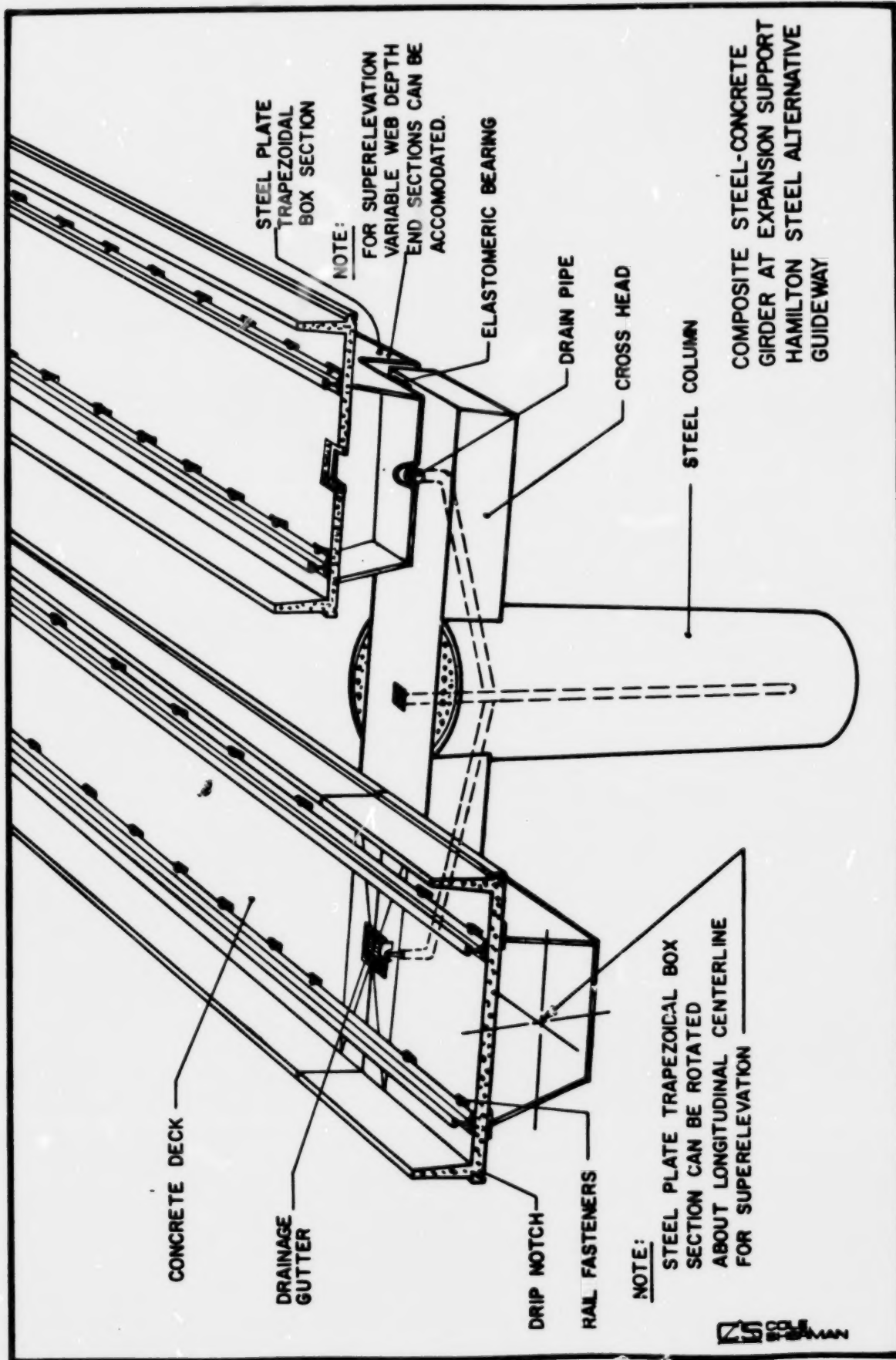


FIGURE : 3

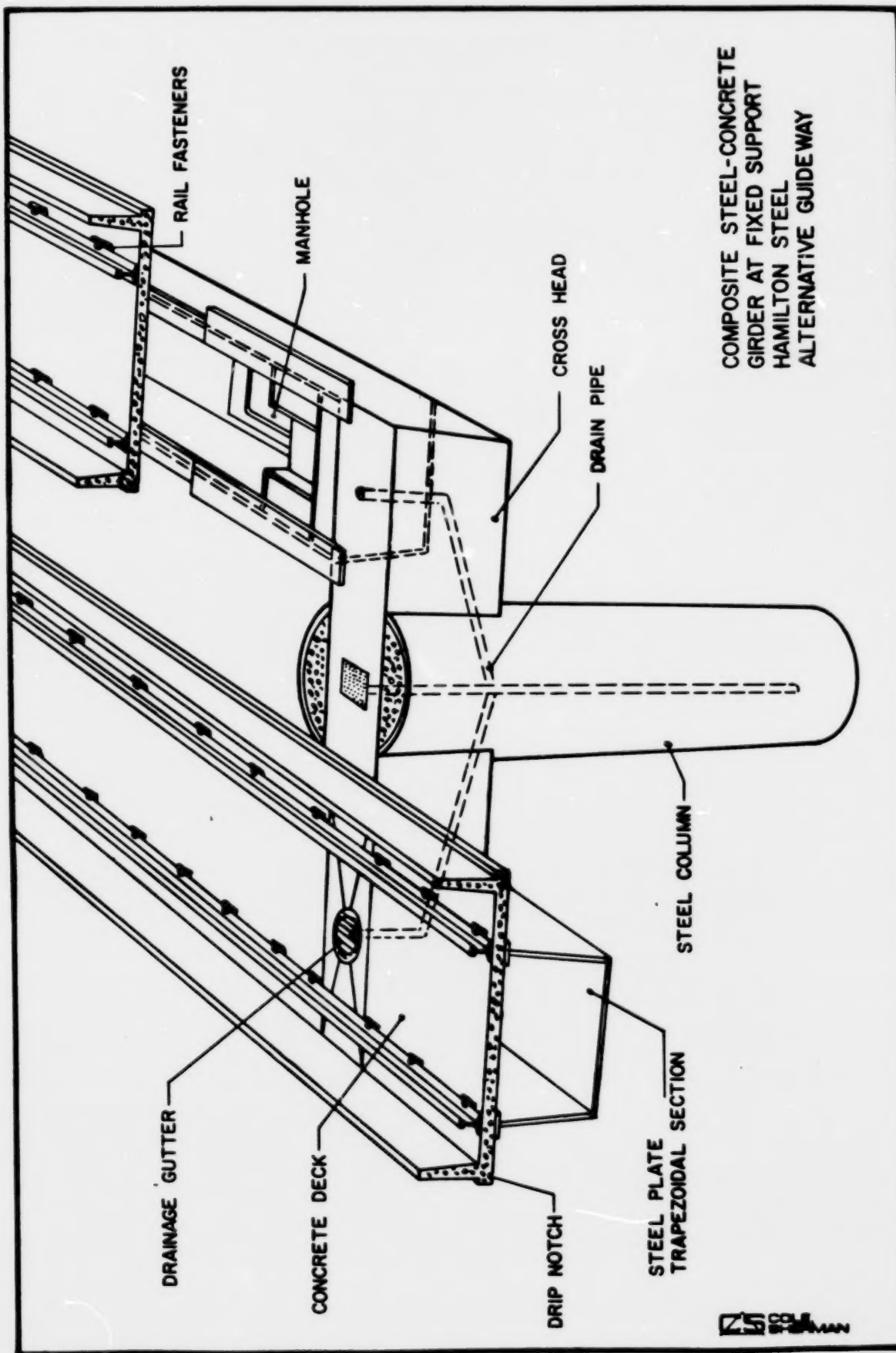
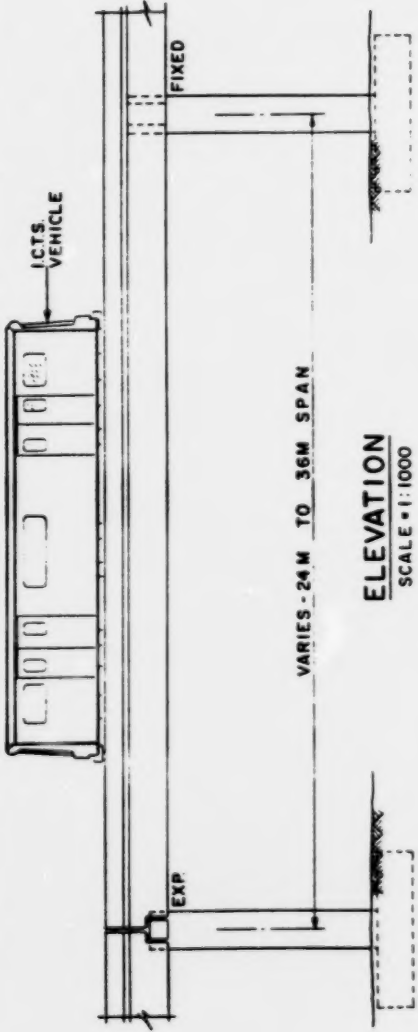
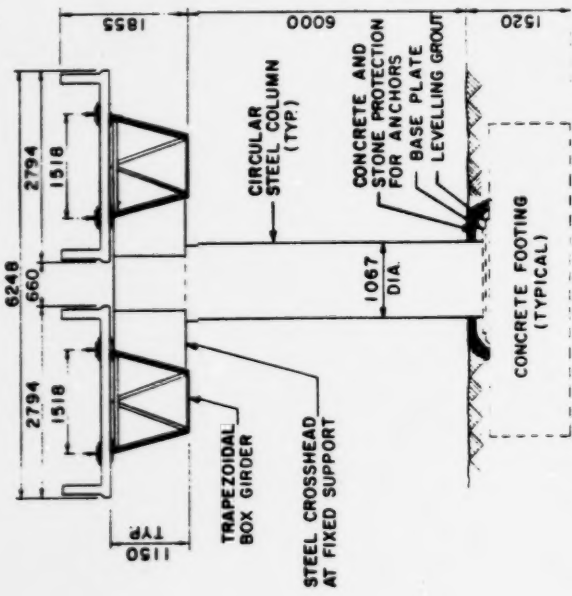


FIGURE : 4

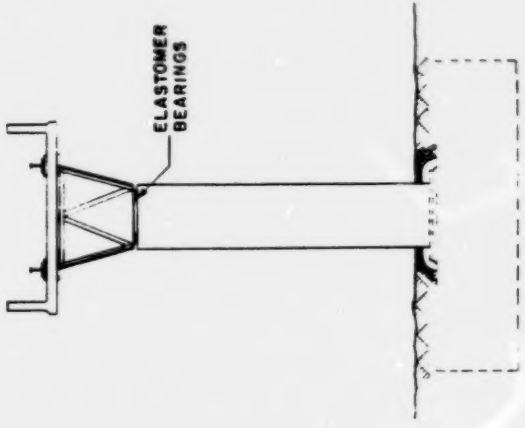


ELEVATION
SCALE = 1:1000

SYMMETRICAL ABOUT PIER

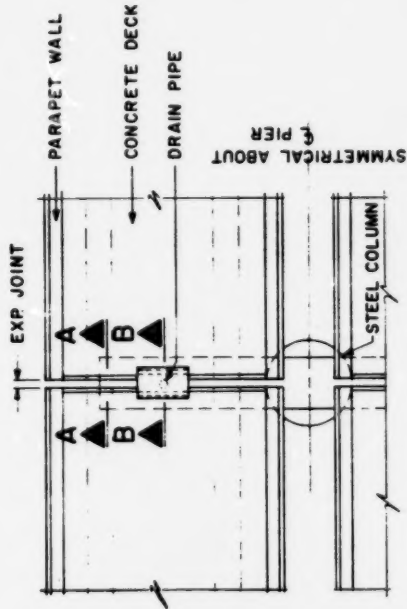


SECTION - DUAL GUIDEWAY
SCALE = 1:500

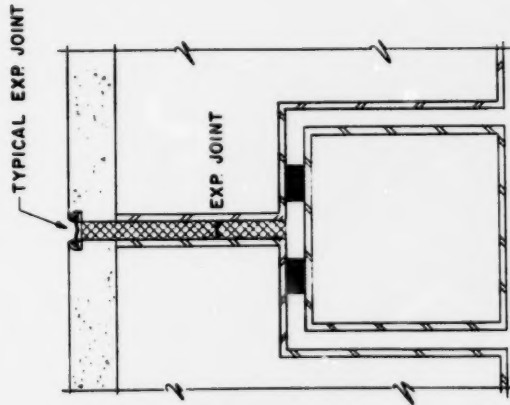


SECTION - SINGLE GUIDEWAY
SCALE = 1:500

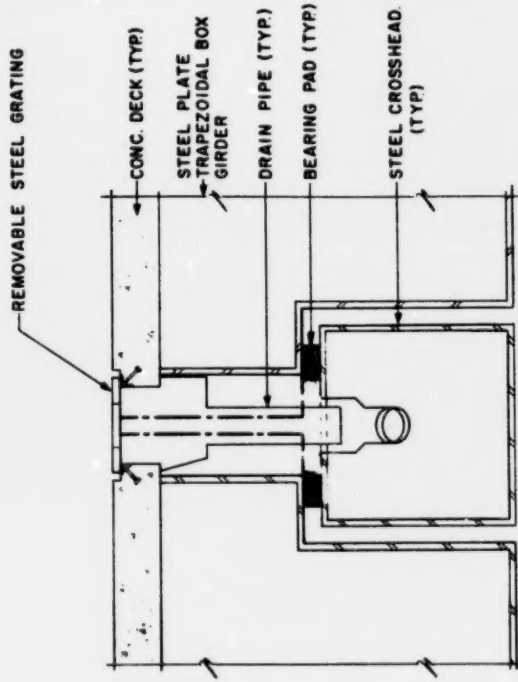
Hamilton Wentworth Rapid Transit Project	STEEL GUIDEWAY CONCEPTUAL LAYOUT	
	Date MAR 1981	Figure: 5



PLAN AT EXP. END
SCALE = 1" = 40'



SECTION A-A
SCALE = 1" = 10'



SECTION B-B
SCALE = 1" = 10'

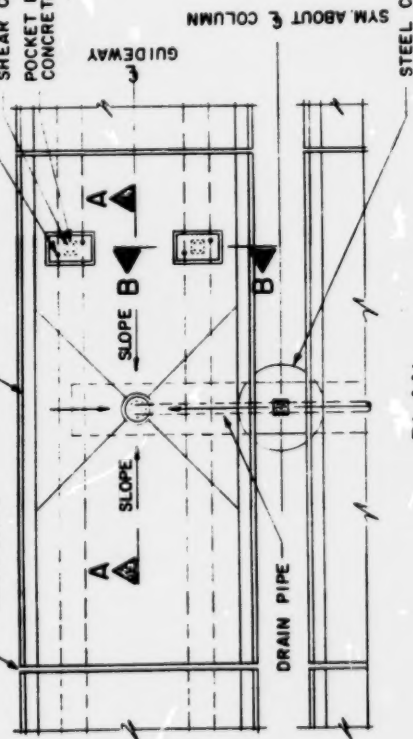
**STEEL GUIDEWAY
EXP. JOINT DETAILS**

Hamilton
Wentworth
Rapid Transit
Project

Date MAR. 1981 Figure: 6

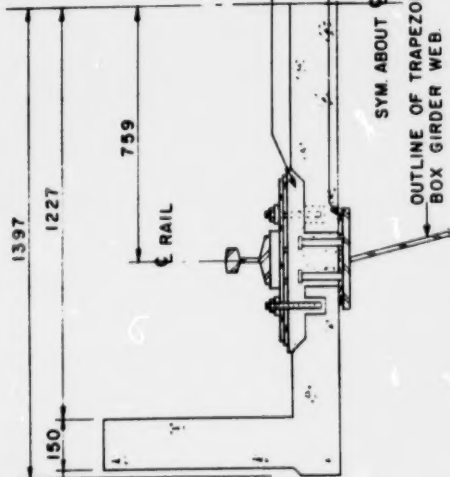
HIXSON RAIL FASTENER
AT 762mm ± c/c
SHEAR CONNECTORS 20
POCKET IN PRECAST
CONCRETE DECK

PARAPET WALL
GROUT BETWEEN
PRECAST CONCRETE
DECK UNITS



PLAN

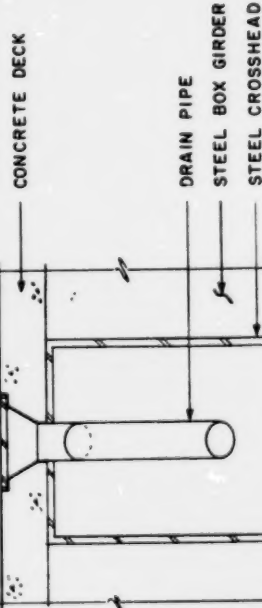
SCALE = 1:40



SECTION B-B

SCALE = 1:10

REMOVABLE DRAIN
COVER PLATE



SECTION A-A

SCALE = 1:10

Hamilton
Wentworth
Rapid Transit
Project

STEEL GUIDEWAY
PRE-CAST DECK DETAILS

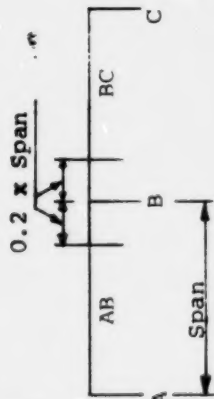
Date MAR. 1981

Figure: 7

SUMMARY OF BOX GIRDER - PROPERTIES

Depth $d = 1150$ mm
 Web = 12 mm
 No Stiffeners
 All Sizes in mm

Constant
 All Spans



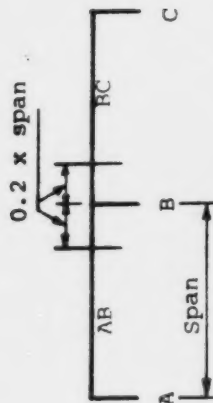
SPAN	DETAILS	AB & BC	B	CROSS HEAD			COLUMN
				A & C	B		
1) 36 m	Top Plate Web Bot. Plate Weight Diaphragms Total Weight Frequency L.L. Deflection	2 - 300 x 22 1130 x 12 950 x 20 4.57 KN/m 10% Box Weight 5.03 KN/m 3.2 > 3.0 Hz. Span/1130 < Span/800	2 x 500 x 35 1115 x 12 950 x 38 7.54 KN/m 10% Box Weight 8.29 KN/m - -	Width 600 Depth 575 Plate Thickness 35	Width 600 Depth 1150 Plate Thickness 35		Diameter 1067 Wall Thickness 16
2) 30 m	Top Plate Web Bot. Plate Weight Diaphragms and Long. Stiffeners Total Weight Frequency L.L. Deflection	2 - 300 x 12 1138 x 12 950 x 12 3.53 KN/m 10% Box Weight 3.89 KN/m 4.2 Hz. Span/1932 < Span/800	2 - 300 x 35 1120 x 12 950 x 30 5.88 KN/m 10% Box Weight 6.47 KN/m - -	Width 600 Depth 575 Plate Thickness 25	Width 600 Depth 1150 Plate Thickness 20		Diameter 1067 Wall Thickness 16

TABLE: 3

SUMMARY OF BOX GIRDER - PROPERTIES

- Depth = 1150 mm
- Web = 12 mm
- No Stiffeners
- All sizes in mm

constant
all spans



SPAN	DETAILS	AB & BC	B	CROSS HEAD		COLUMN
				A & C	B	
3) 24 m	Top Plate Web Bottom Plate Weight Diaphragms and Long. Stiffeners Total Weight Frequency L.L. Deflection	2 - 300 x 10 1140 x 12 950 x 10 3.3 kn/m 10% box weight 3.63 kn/m 6.5 Hz Span/3807	2 - 300 x 25 1130 x 12 950 x 20 4.71 kn/m 10% box weight 5.18 kn/m - -	Width 600 Depth 575 Plate Thickness 20	Width 600 Depth 1150 Plate Thickness 16	Diameter 1067 Wall thickness 16
4) 18 m Curve R=35m *	Top Plate Web Bottom Plate Weight Diaphragms and Long. Stiffeners Total Weight Frequency L.L. Deflection	2 - 300 x 10 1140 x 12 950 x 10 3.3 kn/m 10% box weight 3.63 kn/m 11.9 Hz Span/6345	2 - 300 x 25 1130 x 12 950 x 20 4.71 kn/m 10% box weight 5.18 kn/m - -	Width 600 Depth 575 Plate Thickness 20	Width 600 Depth 1150 Plate Thickness 16	Diameter 1372 Wall thickness 25.4

TABLE 4

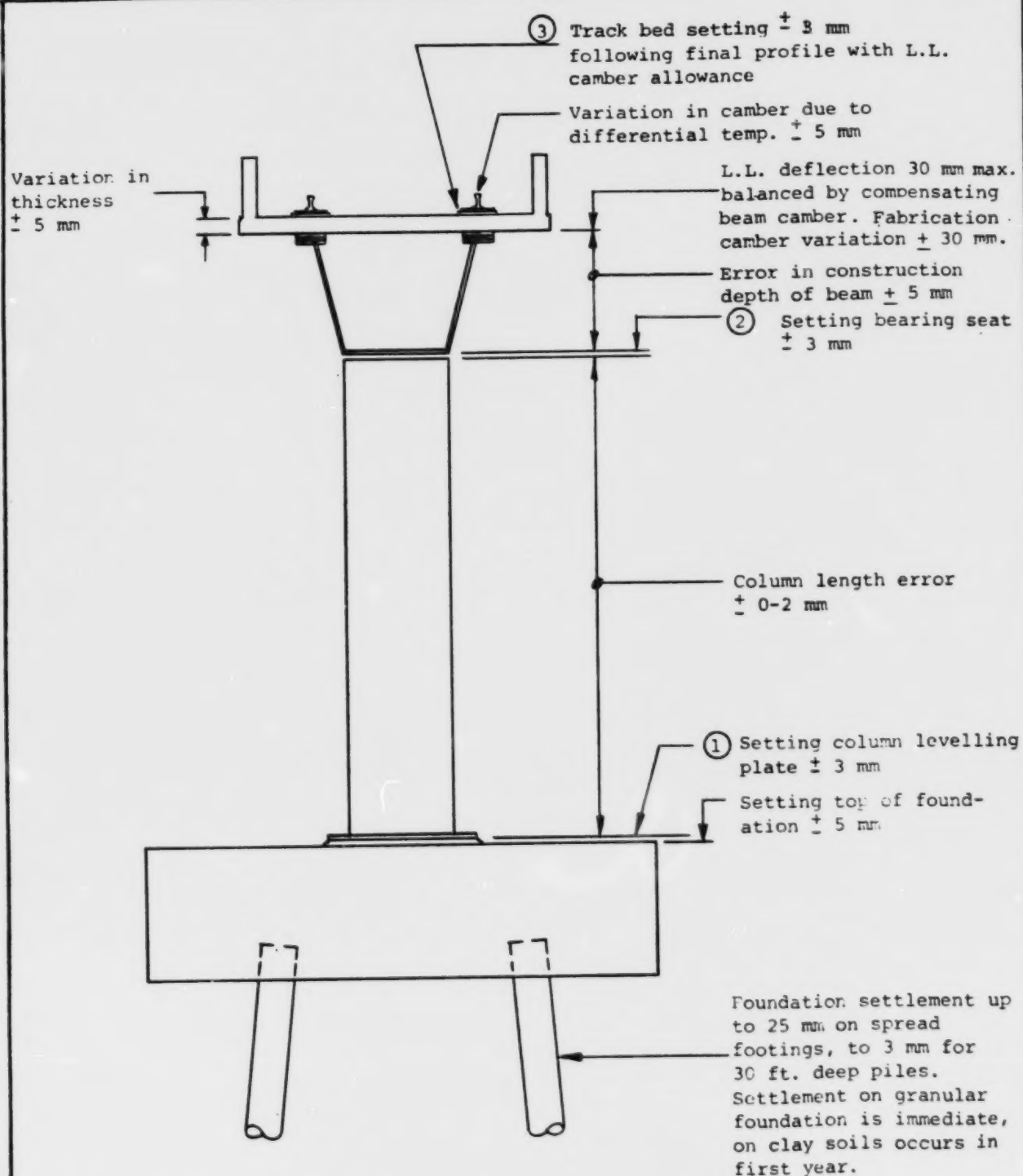
5. TOLERANCES

In order to achieve the high quality of ride specified for the guideway system particular attention has to be paid to manufacturing and construction tolerances. This is especially so at the track support level where final adjustments are made and errors can have a profound effect on rail alignment.

Figure #8a illustrates the main features of the vertical alignment control system for a typical single steel guideway. Similar variables would be found in dual and curved guideways and for horizontal alignment.

At each stage of construction or erection, previous errors and some parts of foundation settlement can be corrected. These stages are numbered 1, 2 and 3. At stage 1, errors in pile cap or footing elevation can be reduced to $\pm 1/8"$ (± 3 mm) from desired elevation. Should foundation conditions change along the guideway from spread footing to rock bearing, some allowance for differential settlement could be made at that stage by setting the levelling plate high on the spread footing. At Stage 2, shimming or grouting of bearing seats, overcomes errors in column length and restores vertical alignment accuracy to $\pm 1/8"$ (± 3 mm). Variations in beam camber, beam depth and thickness of concrete deck can be absorbed at Stage 3 when grouting the shear connectors thereby providing a rail profile which includes an allowance for beam deflection under half vehicle loading. The total variation to be absorbed by shimming at Stage 3 is 43 mm (1-3/4"), part of which can be accommodated by the rail fastener shim and part by shims placed between the precast concrete deck and the top flange of the box beam prior to grouting the shear connectors. No allowance can be provided for effect of daily temperature variation. At the final Stage -3-almost all foundation settlement will have taken place except in the case of clay soils where about 75% of expected movement could take place over the six months following completion of construction. At transition points between zones of varying foundation type, an allowance for this effect can be provided in foundation design to reduce differential settlement.

Alternative construction methods to that illustrated would require a similar analysis for alignment control. These may offer advantages in construction cost, but disadvantages in tolerance control. In this respect, precasting the concrete



VERTICAL ALIGNMENT CONTROL STEEL GUIDEWAY

Figure 8a

deck on the steel box girder on an inverted form, complete with rail fasteners, would eliminate the possibility of absorbing camber errors by a second pour. Beam distortion occurring during shipping and erection, as well as the effect of foundation settlement, would remain to be corrected by rail fastener adjustment only.

This construction system and associated tolerances have been assumed in estimating the construction costs.

Figure 8b illustrates the vertical control system of a typical concrete guideway which can be compared with that for steel.

Up to Stage 2, the compensations for construction error in the concrete substructure are essentially the same as those for the steel. From this point on in the erection sequence there is no built-in mechanism for adjusting track bed elevation except by shimming the rail fastener. This means that the rail fastener shim must absorb the accumulated variations in beam depth, fastener location and camber. Camber can be affected by any of the following factors: forming tolerance, prestressing, post-tensioning, creep due to dead load, and creep due to eccentricity of stressing cables. The value of the creep deflections is affected by the length of time between pouring the concrete and applying the prestress and on other factors such as variations in humidity of the air during curing and also strength of the concrete. Camber and deflection due to differential temperature effects and live load cannot be avoided. The rail fastener shim will therefore be required to compensate for a possible maximum variation of 30 mm (1 1/4"). Differential and absolute settlements of foundations will be the same for both guideway materials and can be accommodated in the foundation design stage.

Variation in camber
due to dead-load, stressing
and creep ± 12 mm

③ Variation in camber due to
differential temp. ± 5 mm

L.L. deflection 30 mm max.
balanced by compensating
beam camber

Error in construction
depth of beam ± 5 mm

② Setting bearing seat
 ± 3 mm

Column length error
 $+ 10$ mm to $- 3$ mm

① Setting column levelling
plate ± 3 mm

Setting top of found-
ation ± 5 mm

Foundation settlement up
to 25 mm on spread
footings, to 3 mm for
30 ft. deep piles.
Settlement on granular
foundation is immediate,
on clay soils occurs in
first year.

VERTICAL ALIGNMENT CONTROL CONCRETE GUIDEWAY

Figure 8b

6. CORROSION DUE TO STRAY ELECTRIC CURRENTS

Electrically powered transit systems have historically suffered from the consequences of stray currents. Those systems that employ DC current and use the running rail as a ground return such as streetcars are particularly vulnerable. Damage takes the form of excessive and rapid corrosion of the ends of metal parts which carry the stray current. These could be the running rail, steel beam, reinforcing in the concrete structure or water pipes running parallel to the track. Corrosion is most dramatic at the ends of the metal items where the current leaves to pass into a material such as soil or concrete, especially in the presence of moisture and industrial wastes.

In the case of the Hamilton Guideway System there should only be an insignificant amount of stray DC current generated by the guideway vehicle and power rail, however, other sources may exist along parts of the route and should be identified at final design stage. The continuous rail proposed for the Hamilton system is grounded at frequent intervals which should minimize any stray current problem.

Sensitive points in the guideway structure would be at the expansion joints in the beams and at the connections between column and foundation, column and beam and pile cap and piles, and also at the toes of piles.

Damage resulting from rusting of reinforcing bars in concrete has been well documented elsewhere and takes the form of splitting and bursting of the concrete due to enlargement of the rust mass as the steel is oxidized. Structural steel members rust more rapidly under the action of stray DC current than under normal conditions.

The usual method of preventing the electrically induced corrosion is to provide the current with a continuous path through the structure which terminates in a sacrificial member at each end (assuming reversals can occur). Provision of electrical continuity is achieved in steel structures by interconnecting the steel beams electrically by flexible copper cables. Reinforcing bars and stressing cables in concrete structures must be electrically connected by welding grounding cables to each bar and cable within the concrete and connecting this

collector by a flexible cable to a similar collector in the next beam. Reinforcing bars and cables within each beam must be electrically continuous or corrosion will occur at lapped splices within the member.

Such protection systems are expensive for reinforced concrete and are best avoided if possible by other means. The elastomeric bearings generally used to support bridge structures provide excellent electrical isolation but metal drain pipes and conduit running down from deck to foundation can provide a route for DC current. Attention to design details in choice of materials and connections for these items can significantly reduce the probability of electrical corrosion problems occurring and should form part of the design process.

Although the frequent grounding of the continuous rail will minimize the corrosion problem it would be prudent to consider providing bonding connections on the ends of each steel box girder for future use or for monitoring of stray currents.

7. AESTHETICS

Architectural input to this section was prepared by Trevor Garwood Jones - Architect.

7.1 Architectural Considerations

The alternate use of steel, in lieu of concrete, for the construction of the guideway beams, crossheads and columns presents several factors for consideration from the architectural viewpoint. For the purposes of this report, only weathering steel is being considered because of either the extremely high initial cost of stainless steel or porcelain enamelled steel, or the continuous maintenance requirements of painted steel.

7.2 Functional Suitability

Weathering steel is well suited from a functional standpoint for use in this application. This material depends on bold exposure to the weather and uniform washing through continuous wetting/drying cycles for colour development. In addition the stable oxide film is generated more readily in the presence of sulphur gases and is therefore more successfully achieved in an industrial, rather than a rural, atmosphere.

The run-off of soluble iron oxides will occur to a major degree during the first two or three years following installation and will continue at a greatly reduced rate for the life of the installation. The staining of adjacent surfaces exposed to this run-off requires careful consideration in detailing in order that it be avoided. The incorporation of gutters, corner drips and suitable drainage beds is mandatory. The successful, 100% elimination of staining due to run-off cannot be guaranteed because it is physically impossible to design for the all-weather exposure, i.e. wind conditions which will be encountered.

The weathering steel surfaces themselves provide the obvious great advantage of being maintenance-free. In addition, due to the dark colour of weathering steel, it is less susceptible than concrete to staining from industrial atmospheric pollution.

It is worthwhile to note that a system of pre-coating and pre-weathering the steel surfaces has been developed and is currently being tested. This could successfully solve the problems associated with run-off to a high degree. Such a system would likely involve an initial cost factor similar to that for site painting.

Weathering steel requires great care in handling during the construction period in order that uniform weathering occurs. Sandblasting following the completion of construction may have to be considered.

It should also be noted that the bottom portion of columns adjacent to grade require careful consideration due to continual exposure to wet conditions and to chloride salts which would prevent uniform weathering.

7.3 Visual Considerations

Following sufficient exposure to appropriate conditions, weathering steel takes on a rich, dark brown, rough-textured appearance. With the incorporation of the pre-weathering method currently being studied, it is possible to generate a final colour more in the dark grey-blue range. The dark colour of weathering steel is more readily associated with the organic colours of nature than, for example, a light beige coloured concrete tone. From a distance, its appearance is far less artificial and its visual impact against a natural environment is less pronounced than against a man-made background. However, at closer range and particularly when introduced into a downtown core setting of concrete, masonry, glass and asphalt, the visual impact of weathering steel becomes industrial and out of place, rather than indigenous with its surroundings.

The overall visual impact of the guideway system, because of its mass and proportions as generated by structural requirements, cannot be negated. The guideway itself is approximately as wide as it is elevated from the grade level and the column height to width proportion will be approximately 6 to 1. Regardless of materials used, assuming transparent plastics or glass not to be feasible, the guideway system will be visually perceived as heavy in mass and blocky in proportion. This will occur to the greatest degree from the pedestrian, close-scale range.

There are visual advantages to using weathering steel in natural and residential surroundings for the guideway columns, beams and crossheads, in conjunction with coloured concrete precast deck and sidealls; and conversely for using coloured concrete for the total guideway system in high-density urban areas.

7.4 Integration - Station Design

Our preliminary, intuitive consideration of station design has generated the idea that urban stations should be light and/or bright in colour and incorporate typical, high-tech materials such as painted or stainless steel and transparent panels in order to best integrate with their surroundings. For the same reason, residential stations will incorporate dark natural colours and rougher textures. The sophisticated, smooth-surfaced, hard-edged appearance of the urban stations would better suit a concrete guideway system and conversely the darker, less-sophisticated rougher materials of the residential stations would better suit a weathering steel guideway system. At this very preliminary stage without an exact guideway alignment, or station locations being finalized, we would present the possibility of a transition from a concrete system to a weathering steel guideway system occurring at the tunnel portal or at other residential/downtown transition points as being visually logical.

We would reiterate our opinion that material colours and textures will influence the visual impact of the guideway system only marginally. Reduction in size and mass, alternate geometric profiles and degree of free buffer surrounding area will have much greater significance.

7.5 Weathering Steel

The steel guideway study has been carried through with the general assumption that weathering steel will be the probable candidate material since this material avoids the looming spectre of high maintenance cost of painting elevated structures. Significant data is reviewed in this section related to its use for the Hamilton Guideway.

In summary, weathering steel is in many ways similar to regularly available structural steels except that its corrosion products adhere to the surface and form an impervious oxygen-proof inert film which prevents further corrosion. It

is self-healing in the sense that if the oxide film is removed by scratching or abrasion, it will re-form. The oxide film is dark brown in colour but will become almost "blue" in industrial atmospheres in time. There have been some concerns arising from the use of steel as a guideway material as there have with any material and unless these are known and remedies applied during the design and construction phases, the material chosen will not perform as intended. For example, untreated timber piles will rot if not protected by chemical impregnation and concrete highway curbs will fragment without air entrainment in the concrete mix.

7.6 Stain Control and Treatment

The weathering steel oxide film forms rapidly during the early months of exposure and during its formation some of the products of oxidation are washed off the surface by rainwater. These iron oxide products can severely stain the surfaces of such objects as concrete columns, sidewalks, brick, etc., and the construction period is clearly very critical in this respect. The production of a satisfactory appearance and avoidance of staining can be equated to a problem of water management. Water collecting inside the exposed box girders, for example, and washing down concrete piers can be easily avoided by built-in temporary drainage systems. With well thought out construction scheduling it can be arranged that construction of the concrete box girder lid can follow closely behind girder erection, thus providing protection from runoff. Drainage holes with drips must be provided in bottom flange of box girders to allow leakage water to escape from inside. Expansion joints can be provided with a leakage collection drain and the concrete can be protected from staining.

Concrete columns have been protected by a wide range of materials including polyethylene sheets during construction, silicone coatings and clear copolymer coatings. Polyethylene sheets can tear and require maintenance. Silicone coatings prevent the formation of permanent stains by inhibiting the chemical integration of the iron oxide into the concrete, but seem to permit temporary surface staining which can be removed by a weak solution of muriatic acid. Of the 60 clear coatings investigated by the Portland Cement Association the copolymer based on methyl methacrylate offered the best protection for concrete. This type of coating would last six to ten years after which runoff staining would be minimal.

In this context, Nippon Steel of Japan has developed a exterior surface treatment for weathering steel which prevents runoff of iron oxide during formation of the impervious coating. This begins with cleaning of the steel surface then applying a proprietary solution and finally, spraying with an acrylic resin. Oxidation of the steel proceeds below the protective acrylic film which eventually weathers away leaving the usual weathering steel surface. Stelco is examining the viability of this treatment for the Canadian market.

Attention must be paid in the design stage to detailing of the steel girders to avoid pockets or other areas where debris can collect and stay wet. In this circumstance, weathering steel continues to corrode and does not form a permanent protective coat. Designers in the steel industry are well aware of these requirements.

The use of weathering steel columns minimizes the runoff staining consideration. Attention to detail is again required to avoid the wet pocket problem.

The oxide film formation is more rapid in an industrial atmosphere due to the greater availability of sulphur dioxide. This is an advantage from the long term point of view, but a disadvantage during construction.

As with any structure, an on-going inspection program is required. In the case of weathering steel, this would establish that corrosion had proceeded as expected and that water management was maintained.

8. CONCRETE - STEEL COMPARISON

Through the co-operation of C.I.S.C. and CSICC unit prices have been obtained from the steel industry for the components of the steel guideway alternative. These are listed in Table 6 and are valid for the beginning of 1981. Unit prices have also been obtained from local contractors for the concrete deck and column fill for both precasting and formed-in-place methods of production. The unit prices for the concrete guideway, Table 5, were obtained from ABAM Limited except for those of foundations which were developed by Cole, Sherman & Associates Limited, Hamilton. To these values have been added the cost of vibration damping, provision for drainage and electrical conduit, bearings, expansion joints, the installation of rail fasteners and final sandblasting and cleaning of steel surfaces. Running rail and fasteners, LIM rail, LIM reaction rail control cable mounting fixtures, drainage and electrical conduit have all been excluded. All prices include General Contractor's overhead and profit of 15%, a contingency allowance of 5-10% and an allowance for 5% Federal Tax on materials. Materials have been assumed to constitute 50% of the total cost for the level of accuracy of this analysis.

Table 7 contains the cost breakdown for each element of guideway consisting of beam, column, and foundation. For the sake of this comparison it has been assumed that single guideway foundations require 34 m deep piles and that foundation costs are the same for the concrete and steel guideways. All tangent spans have been assumed as 30 m in length and in the case of 35 m radius curve as 18 m length. Average foundation costs have been used for dual guideway. Table 8 provides a visual display of the unit element costs.

The calculated unit costs for each construction element show that, for tangent section, concrete and steel are within 6% of each other. However, on 35 m radius curves, steel clearly has the advantage being about 20% cheaper than concrete. In the calculation of the cost for large radius curves, an average price between 35 m and tangent section was used. The fabricators have stated that the cost difference between tangent section and large radius curves is minimal. Steel still has a significant advantage in this configuration being about 15% cheaper than concrete.

A. CONCRETE BEAM PRESTRESSED AND POST-TENSIONED (30 m SPAN)

1. Single Tangent Alignment	\$ 1,250/m
2. Single 35 m Curve Alignment	\$ 2,310/m
3. Dual Tangent Alignment	\$ 2,310/m
4. Dual 35 m Curve Alignment	\$ 4,370/m
5. Dual 300 to 35 m Curve Alignment	\$ 3,340/m

B. COLUMNS AND CROSSHEADS (6 m HIGH)

1. Single Tangent	\$ 160/m
2. Single 35 m Curve	\$ 215/m
3. Dual Tangent	\$ 215/m
4. Dual Curve	\$ 270/m

C. FOOTING

	Pile Lengths	Tangent	35 m Curve
1. Single Guideway	24 m	\$ 11,800/ftg.	\$ 38,400/ftg.
	43 m	\$ 16,600/ftg.	\$ 52,800/ftg.
	18 m	\$ 9,000/ftg.	\$ 30,000/ftg.
2. Dual Guideway		\$ 14,000/ftg.	\$ 18,000/ftg.

UNIT PRICES - CONCRETE

Table 5

A. TRAPEZOIDAL STEEL BEAM BOX (30 m SPAN)

1. Single Tangent Alignment	\$ 781/m
2. Single 35 m Curve Alignment	\$ 886/m
3. Dual Tangent Alignment	\$ 1,562/m
4. Dual 35 m Curve Alignment	\$ 1,772/m
5. Dual 300 to 35 m Curve Alignment	\$ 1,667/m

B. COLUMNS AND CROSSHEADS (6 m HIGH)

1. 1,067 mm dia. x 16 mm Col.	\$ 3,147 each
2. Expansion Crossheads	\$ 4,858 each
3. Fixed Crossheads	\$ 5,907 each

C. FOOTING

	Pile Length	Tangent	Curve Sec.
1. Single Guideway	24 m	\$11,800/ftg.	\$38,400/ftg.
	43 m	\$16,600/ftg.	\$52,800/ftg.
	18 m	\$ 9,000/ftg.	\$30,000/ftg.
2. Dual Guideway		\$14,000/ftg.	\$18,000/ftg.

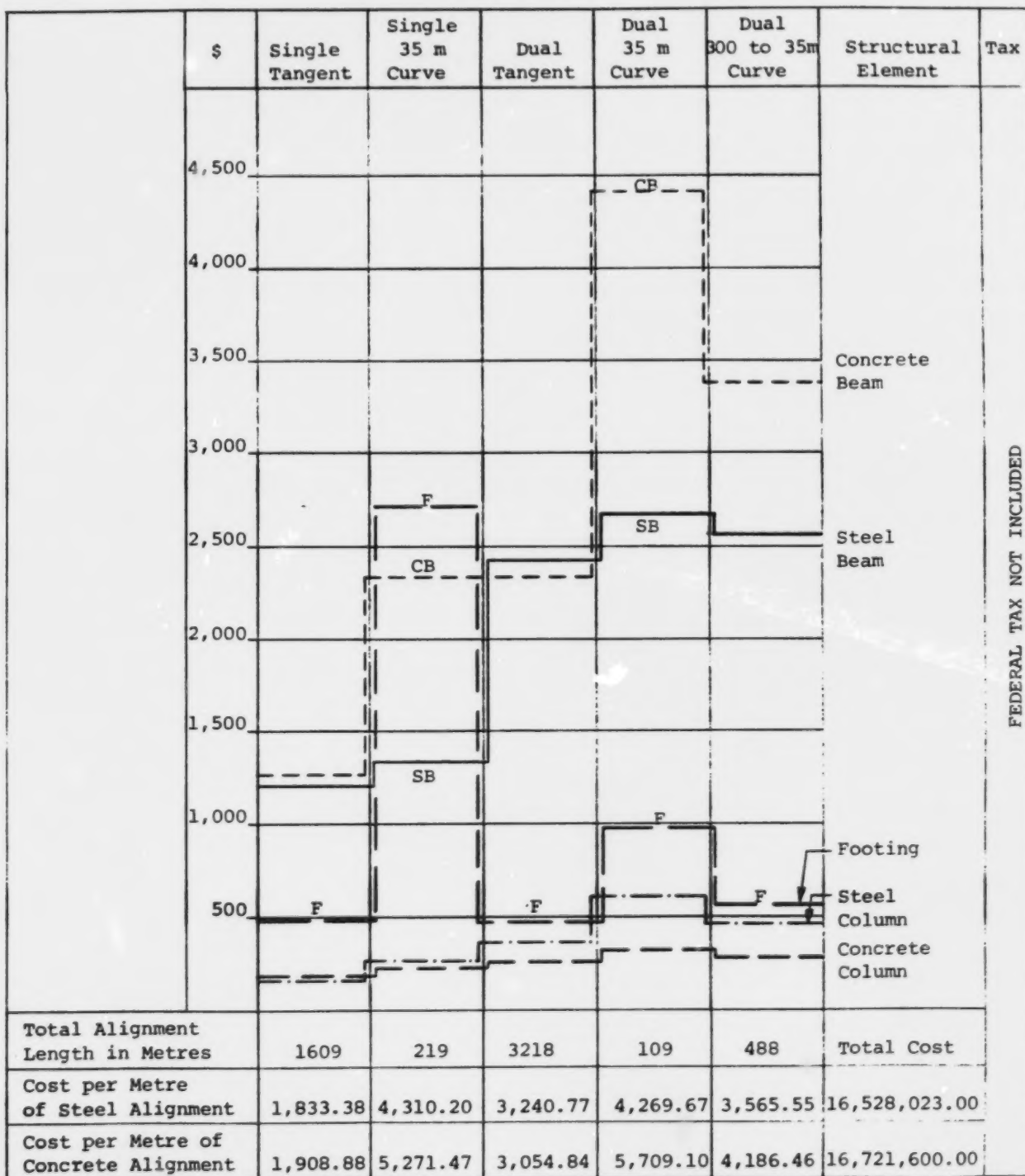
UNIT PRICES - STEEL

Table 6

I T E M S	SINGLE TANGENT		SINGLE 35 m CURVE		DUAL TANGENT		DUAL 35 m CURVE		DUAL 300 TO 35 m CURVE		TUNNEL
	STEEL	CONCRETE	STEEL	CONCRETE	STEEL	CONCRETE	STEEL	CONCRETE	STEEL	CONCRETE	
BEAM											
Beam (Naked)	781.00	1,250.00	886.00	2,310.00	1,562.00	2,310.00	1,772.00	4,370.00	1,667.00	3,340.00	
Damping (100% of alignment)	211.28		211.28		422.56		422.56		422.56		
Precast Deck	194.40	12.40	220.84	21.84	388.70	24.70	438.60	40.60	423.64	25.64	
Sandblast and Cleaning	21.70		21.70		43.40		43.40		43.40		
A BEAM TOTAL/m	1,208.38	1,260.40	1,339.82	2,331.84	2,416.66	2,334.70	2,676.56	4,410.60	2,556.60	3,365.64	16,154.68
COLUMN											
Steel for Col. dia. = 1067 mm dia. = 1372 mm	105.64	160.00	186.81	215.00	286.26	215.00			377.00	242.50	
Concrete for Col.	27.16		49.15		27.17		469.51	270.00	47.00		
Bearing pads, etc.	9.93	9.91	17.78	17.78	35.14	35.14	86.02	37.50	20.92	20.92	
Sandblast and Cleaning	5.54		9.79		5.54		9.08		5.74		
B COLUMN TOTAL/m	148.43	169.91	263.53	232.78	354.11	250.14	602.11	307.50	451.55	263.42	
Footings	476.53	476.57	2,706.85	2,706.85	470.00	470.00	991.00	991.00	957.40	557.40	
FOOTING											
C FOOTING TOTAL/m	476.57	476.57	2,706.85	2,706.85	470.00	470.00	991.00	991.00	557.40	557.40	
D TOTAL COST/m A + B + C	1,833.36	1,908.88	4,310.20	5,271.47	3,240.77	3,054.84	4,269.67	5,709.10	3,565.55	4,186.46	16,534.68
E 2% Federal Tax D x 0.025	45.83	47.72	107.75	131.79	81.02	76.37	106.74	142.73	89.14	104.68	408.87
F TOTAL COST/m D + E	1,879.22	1,956.60	4,417.95	5,403.26	3,321.79	3,131.21	4,376.41	5,851.83	3,654.69	4,291.12	16,763.55
G Length of Each Section in m	1609	1609	219	219	3218	3218	109	109	488	488	609
H TOTAL COST F x G	3,023,649.00	3,148,169.00	967,531.00	1,183,314.00	10,689,520.00	10,076,234.00	477,029.00	637,850.00	1,783,489.00	2,094,067.00	10,209,002.00

ALIGNMENT COSTS - STEEL & CONCRETE

Table 7



FEDERAL TAX NOT INCLUDED

COMPARISON OF UNIT CONFIGURATION COST

Table 8

To bring the unit configuration costs into perspective they have been applied to a typical guideway alignment which contains the following lengths of guideway:

Single tangent section	-	1609 m
Single 35 m radius curve	-	219 m
Dual tangent section	-	3218 m
Dual 35 m radius curve	-	109 m
Dual 35 m to 300 m curve	-	488 m
Tunnel (added for completeness)	-	609 m

Based on these lengths a total construction cost has been estimated. The estimated cost of the steel alternative guideway alignment is close to that of the concrete guideway. The steel alternative appears to offer only 1% saving in cost compared with concrete.

This benefit could be improved with greater refinement of the steel design. For example a 10% gross deadload allowance was assumed in the design for sound damping materials and it has subsequently been ascertained that only 3% is required. In addition, the box beam shape and size was developed with the built-in flexibility of its being extended to a span of about 37 m. Should this facility prove unnecessary, a further 5% weight reduction could be obtained in the steel box girder.

The steel fabricators have stated that box girder fabrication costs could be reduced by 10% in the case of tangent section and 18% in the case of curved guideway section if vertical sided boxes are used. These values would be halved in terms of the final subcontract cost but are worth considering.

In general the total cost of a facility such as this elevated transit structure is only partially represented by the contract price. Cost to the community as a whole includes the inconvenience suffered during construction due to temporary street closures, dust from excavations and concrete placement and grinding and also noise and fumes from such activities as pile driving and from construction equipment in general. The extent of construction operations and the duration of the contract will be affected to some extent by the choice of construction

#	PROPOSALS	1		2		3		4		5		6		7		8		9		10	
		Total Cost of Proposal Without Tax		Federal Tax - 2 1/4%		Total Cost of Proposal With Tax		Tunnel Cost Without Tax		Federal Tax - 2 1/4%		Tunnel Cost With Tax		TOTAL Project Cost		% of Conc. Prop.					
A	Steel Alignment	16,528,023		413,200		16,941,224		9,960,000		249,000		10,209,000		27,150,224		99.2745					
B	Conc. Alignment	16,721,600		418,040		17,139,640		9,960,000		249,000		10,209,000		27,348,640		100					

Notes:

- 1) Steel Alignment - Steel box, precast conc. lid, steel columns and footings for all sections, plus tunnel.
- 2) Concrete Alignment - Pre-stressed, post-tensioned conc. beam sections, conc. column, and footing for all sections, plus tunnel.

COMPARISON OF TOTAL PROJECT COSTS OF STEEL AND CONCRETE ALIGNMENT

Table 9

materials and a comparison between a concrete and steel guideway can best be made after the parameters associated with the construction of each have been identified.

Typical construction schedules have been assembled for the typical guideway alignment described above, assuming concrete box girders in one schedule and steel box girders in the other. These are shown on Figure 10 and were prepared from data supplied by the construction industry as regards the rate of production and erection. They are, of course, based on assumptions which may not apply in an actual real life situation, but it is, in our opinion, valuable to discuss their content as a means of highlighting those parameters which are influenced by the choice of superstructure type.

Regardless of which alternative is chosen the construction will proceed from award of contract through mobilization to piling, foundation construction, column erection, beam erection and rail installation.

Soon after the award of a general contract (if this method is chosen) fabrication and construction of the superstructure would begin. For a concrete guideway this means construction of beam and column forms and ordering of stressing materials. The number of forms manufactured depends on the agreed sub-contract time and will clearly increase the sub-contractor's unit cost if this time is reduced. Fabrication of a steel superstructure requires that adequate notice of the contract be given to the steel industry for the placing of mill orders for steel material which generally must be made three months in advance.

Fabricators prices can be minimized by careful planning which ensures that their shops are operated at optimum efficiency and to achieve this end, advanced notice of forthcoming contracts is a prerequisite.

The sequence of operations required to manufacture and install each of the alternate guideways is shown on the following pages.

Comparison of manufacture and erection

Concrete box girder manufacture is preceded by the assembly of adjustable steel forms the shape of which can be modified to suit the desired beam profile. A sacrificial or removable internal box form is used to form the hollow box shape. Reinforcing bar cages and post-tensioning cable are prefabricated in advance of setting the forms for concreting to maximize use of the casting bed. As soon as the form is set up and rebar installed, concrete can be poured. The following day the pre-tensioning cables can be released and the beam removed from the casting bed if the concrete strength is adequate. A production rate of about twelve beams per week with three sets of forms is expected. Production rate is limited by the need to wait for the concrete to reach the required strength before cables can be released and the beam lifted off its casting bed. Creep deflection control of the beams can be improved considerably by increasing the amount of prestress to give a uniform compressive stress in the concrete after erection.

Trapezoidal precast box girders are rarely, if ever, used in bridge construction today due to their high cost and have been superseded by the lighter I girder. Temporary beam support structures will be required at the fixed piers of the guideway to enable the concrete beams to be made continuous for live load and these will remain in place until post-tensioning is completed. The erection time for continuous concrete girders is lengthened by the need for curing the column grout, the site-cast crosshead joint and the installation of post-tensioning cables. Erection of adjoining spans would have to wait for completion of the post-tensioning operation unless this is done from the deck surface. Time required for column grout to set, beam closure to be cured and post-tensioning operation to be completed, adds sixteen to seventeen days to the construction period.

Each span of beam weighs about 100 tons and is 30 m long, this is close to the upper limit of recommended maximum weight and length for shipping without a police escort suggested by MTC guidelines. If the heavier beams exceed the maximum allowable weight allowance, the concrete side walls would have to be poured in place increasing the cost and slowing construction down. Shipping will probably be impossible during the half load season restricting operations somewhat. Curved beams will inherit the same difficulties as the tangent beams

with increases in forming and shipping cost. Shipping weight is over the 50 tons limit and overall width 4 m which is in excess of the usual 2.5 m permitted on highways. Manoeuvring a curved 50 ton precast box girder in the built-up areas of Hamilton will be a difficult project in itself and temporary tower supports will probably be required at mid-span during the connecting and post-tensioning phase, a period lasting about one week per beam. Precast concrete column production and erection should present no unusual difficulty. A six metre high column weighs about 30 tons which is heavier than the typical precast columns in general use in buildings and may be cheaper to construct in place on the site using circular forms.

The total cost of the precast concrete work incorporated in a typical concrete alignment represents only about 20% of the annual production capacity of the South Ontario precasting industry. This capacity could also be generously extended by some of the local general contractors should they choose to set up their own precasting yard for the project.

In contrast to concrete, the production of steel box girders is relatively straightforward. Their use is common enough for some of the fabricators to have found it worthwhile to construct jigs especially for their manufacture. In any case the tangent girders readily lend themselves to automated welding techniques and employ well tried fabrication methods which form part of a developed production technology. There are many examples of steel box girder structures in Southern Ontario and they are frequently able to compete successfully with the economical precast I beams. The concrete filled columns would be manufactured from pre-fabricated steel tube. Crossheads would be formed from simple flat plates with minimum wastage, cutting and welding. Erection of the box girders will not require any temporary bents and bolted splices can be made directly while the relatively light beams (14 ton) are held by the crane. Shipping will be greatly simplified by the lightweight (14 ton) and shorter length (24 m max.), and could proceed during the half load season and throughout the year. Erection can proceed continuously along the guideway at the rate of two girders per day without interference with street use except during actual lifting. Erection of the precast concrete guideway deck, in 6 m lengths, each weighing 10 tons, can follow immediately behind girder erection as can grouting of shear connectors and joints.

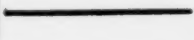
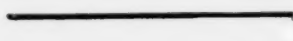
Consideration of the rates of production and erection suggested by the manufacturers has given rise to the times given on the construction schedules. It appears from these that the steel alternative offers only a two week advantage in terms of shortness of the construction period. This is due to the time required for curing the first column grout and start of erection and the last beam closure connecting together the precast girders. It also assumes that the steel industry receives adequate notice of the letting of construction contracts for placing mill orders.

Fabrication of curved trapezoidal box girders will be considerably more expensive (almost double) than that of straight girders due to the wastage of material caused by cutting curved plates from rectangular stock and due to the cost of bending the web plates. In addition the number of splices and bracing has to be increased both inside and outside the girders to control torsional deflections of the open topped box during construction. Curved 18 m. girder spans, however, weigh only 10 tons and being only 2.5 m wide are easy to lift into place in the congested urban areas of the city.

The total weight of structural steel superstructure in the typical guideway alignment is about 4,500 tons which represents about 4.5 percent of the combined annual production capacity of the six Ontario steel fabricators prominent in the construction of bridges and buildings. Supply and delivery for a 12 month construction period should not present any difficulty for this industry. It is unlikely that the cost of casting the concrete deck in place will prove to be less than precasting it unless an economical moveable form can be designed for this purpose. Precasting offers the advantage of speed of manufacture - units can be prefabricated and stored in advance of girder fabrication. However, a cast-in-place deck slab would prove advantageous in crossover areas as it can be poured between the two box girders connected by short stringer beams (10 WF at 2 m centres would be adequate). Non-standard construction, would in this instance, be limited to the deck. The weight of sidewall no longer required at the crossover is equivalent to that of the additional deck slab between girders so that standard steel box girders could be used. Additional elastomeric material could be placed below the rail fasteners at crossovers by using up the levelling tolerance built into the precast deck system to absorb the additional noise generated.

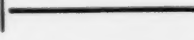
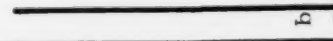
There are several precasters with manufacturing plants near Hamilton, but the Hamilton steel fabricators will have the advantage of local knowledge and short hauling distance which will tend to reduce costs and enable them to retain their competitive position.

CONSTRUCTION SCHEDULE - STEEL

ITEMS	DURATION OF WORK	DAYS
MOBILIZE		10
EXCAVATION		110
PILING		110
PILE CAPS		110
COLUMNS & CROSS HEAD		180
BEAM ERECTION		180
PRECAST SLAB, GROUT JOINTS & INSERTS		180

Accumulative Total = 210 Days

CONSTRUCTION SCHEDULE - CONCRETE

ITEMS	DURATION OF WORK	DAYS
MOBILIZE		10
EXCAVATION		110
PILING		110
PILE CAPS		110
COLUMNS AND CROSS HEAD		190
BEAM ERECTION, SITE CAST JOINT & POST TENSIONING		190

a = Commencement of Work
b = Finish of Work

Accumulative Total = 220 Days

CONSTRUCTION SCHEDULE

Figure 10

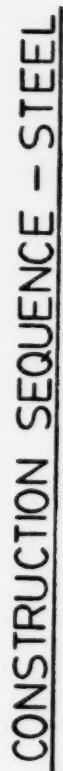


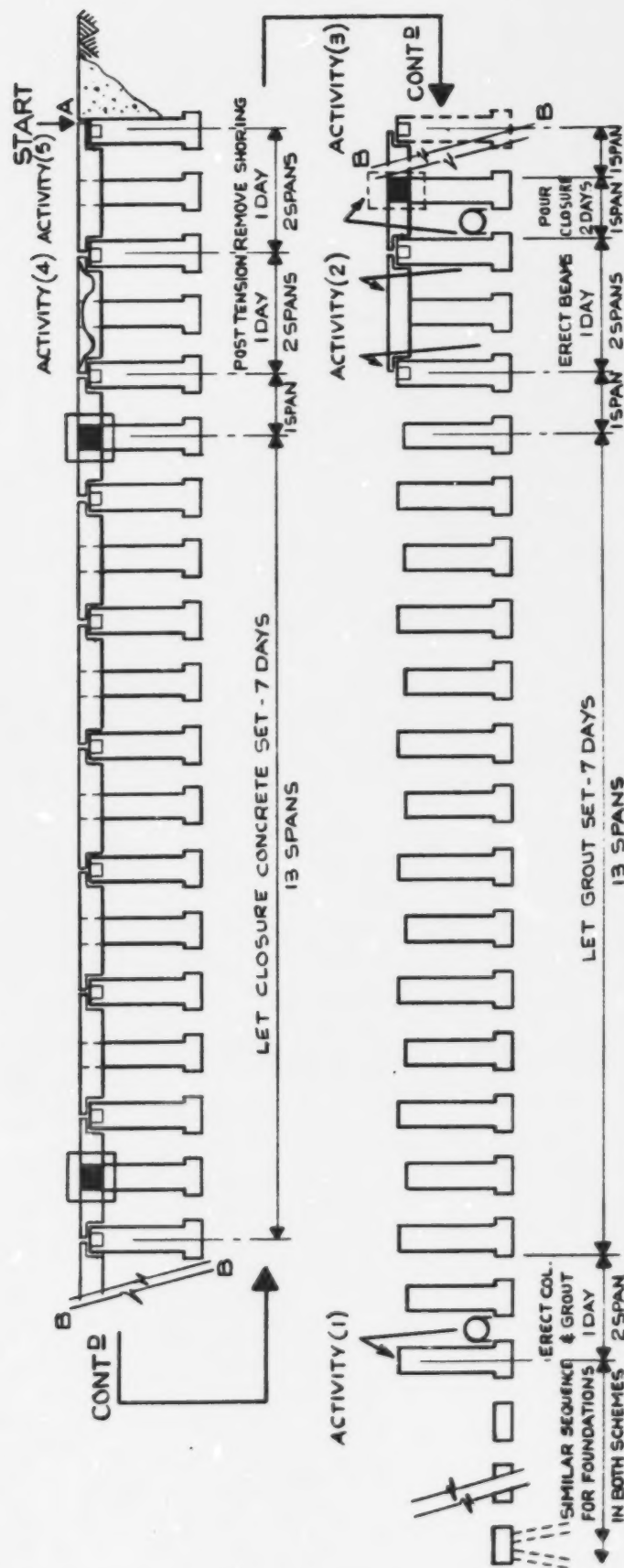
Fig. 11

CONSTRUCTION SEQUENCE - STEEL

Sr. No.	Item of Work	Description of Activity	Urban Impact
1.	Preparation	Shop drawing approval	None
2.	Mobilization	- Set up equipment in shop - Fabrication of steel box beams and columns complete with temporary bracing and sound damping	None
3.	Transportation of fabricated units or materials	Route selection determined by truck load, size and weight. Suggested erection breakdown: a) Ship two columns with cross-heads for erection. No route restrictions due to weight or size b) Ship box beam cantilevers No route restrictions due to weight or size c) Ship two box beams Route determined by length only d) Ship P.C. slab with side walls No route restrictions due to weight or size	Two columns, 4 tonnes each Size: 2.8 m wide x 7 m long each One truck load One beam, 6 tonnes each Size: 1.5 m wide x 12 m long each One truck load Two beams, 12 tonnes each Size: 2 m wide x 24 m long each Two truck loads Ten pieces, 8 tonnes each Size: 2.794 m wide x 6 m long each Two truck loads

CONSTRUCTION SEQUENCE - STEEL (Cont'd)

Sr. No.	Item of Work	Description of Activity	Urban Impact
4.	<u>Site Work</u>	• Clearing of site for foundation	Steel and concrete - same impact
		• Drive piles and pour shaft	1 to 2 days
		• Excavation for column footing	2 to 5 days
		• Form footing	1 to 2 days
		• Place reinforcement	1 to 1 day
		• Pour footing	1 to 2 days
	See 3, above	• Allow footing to cure, strip the forms	Steel and concrete - same impact
		• Ship and erect column with crosshead and grout base plate	1 to 1 day
		• Fill column with concrete	Erection crew with small crane (10 tonnes)
	"	• Ship and erect beams and splice by bolting	Concrete truck, compressor and concrete crew with small crane
		• Ship and erect P.C. slab with sidewalls including grouting joints and pockets	Erection crew and crane (20 tonnes) Pneumatic bolting of 3 connections, short duration
	See 3, above	Total	Erection crew with crane (10 tonnes)
		12 to 19 days	Site activity at one location 240 m length of guideway in any one day



SITE ACTIVITIES AT FIVE LOCATIONS OF 60M AGGREGATE LENGTH EACH.
TOTAL LENGTH OF ACTIVITIES IS 1140 M OF GUIDEWAY IN ANY ONE DAY.

CONSTRUCTION SEQUENCE – CONCRETE

Fig. 12

CONSTRUCTION SEQUENCE - CONCRETE

Sr. No.	Item of Work	Description of Activity	Urban Impact
1.	Preparation	Shop drawing approval	None
2.	Mobilization	- Fabrication of formwork - Precasting of columns - Precasting of box beams 3-Forms used 4 beams/week/form (2.4 beams/day, avg. production)	
3.	Transportation of fabricated units or materials	Route selection restricted by truck load, size and weight. Suggested erection breakdown a) Shop two precast concrete columns b) Ship two P.C. box beams	
			Two columns, 28 tonnes each Size: 1.5 m wide x 6 m long each Two truck loads Two beams, 93 tonnes each Size: 2.75 m wide x 30 m long each Two truck loads Restricted route and police escort required

CONSTRUCTION SEQUENCE - CONCRETE (Cont'd)

Sr. No.	Item of Work	Description of Activity	Urban Impact	
4.	<u>Site Work</u>	• Clearing of site for foundation	1 to 2 days	Same Impact - Steel & Concrete
		• Drive piles and pour shaft	2 to 5 days	
		• Excavation for column footings	1 to 2 days	
		• Form footing	1 to 1 day	
		• Place reinforcement	1 to 2 days	
		• Pour footing	1 to 1 day	
		• Allow footing to cure, strip	1 to 2 days	
		• Ship and erect column and grout	1 to 1 day	
		• Let grout set	7 to 7 days	
		• Ship and erect P.C. beams	1 to 1 day	
	See 3, above	• Form and pour closure, cross-head and side walls	1 to 2 days	One concrete crew and equipment - compressor, concrete truck and small crane
		• Let concrete cure	7 to 7 days	
		• Post tension and grout	1 to 1 day	
		• Remove shoring and open to traffic	1 to 1 day	
	Total	27 to 35 days	Stressing crew and equipment Small gas operated pump	
			Site activities at five locations of 300 m aggregate length - total length of 1140 m of guideway in any one day	

9. FUTURE CONSIDERATIONS AND RECOMMENDATIONS

Preliminary calculations extrapolating the design of the 36 m span indicate that the section could be stretched to 37 m without stepping outside the governing design criteria of live load deflection and frequency of vibration. Incorporation of this span length into the typical alignment could save costs at critical sections such as street intersections where longer spans are advantageous.

The whole topic of continuous rail forces on the columns and foundations of curved guideway and the associated problem of 30 m piles requires careful re-evaluation. Development of these forces is, to a considerable extent, dependant on the rigidity of the guideway structure and its foundations. Analysis should be extended to arrive at an optimum solution which satisfies the demands of economy and structural efficiency.

The design criteria should be examined in the light of recent extensive research and development carried out by MTC in preparing the Ontario Highway Bridge Code. There seem to be solid grounds for modifying the live load factors for this special vehicle and for the development of a more comprehensive approach to such parameters as rider comfort, frequency of vibration and vertical deflection and acceleration than the simplistic 3 Hz and span/800 criteria found in some of the design references. In both the concrete and steel alternative designs the 3 Hz extension is a penalty which, if reduced, would result in cost savings.

Consideration should be given to using vertical sided steel box girders, especially around sharp radius curves perhaps providing transition beams between the trapezoidal shape and the rectangular. Visually it is not easy to tell whether the box web is vertical or sloping and a worthwhile cost saving will result.

Further investigation may prove advantageous in the area of crossover noise generation. Steel seems to offer a viable alternative to concrete if damping is made adequate.

An examination should be made by the steel industry into the possibility of using box girders formed from bent plates to provide a more pleasing appearance to straight section beams and possibly reduce fabrication cost. These would be particularly advantageous for the shorter spans.

Test data on existing steel box guideways may indicate whether sharp radius bends transmit less noise, due to the curved plates, than tangent section. This should be evaluated for sound damping.

The introduction of a permanent mid-span diaphragm between the steel box girders of dual guideway will significantly reduce the torsional moments in the fixed support crosshead and will substantially improve the ride comfort. The diaphragm would be fairly inconspicuous as it need not be more than half the depth of the box girders. Similar diaphragm beams would be valuable in the sharp radius bend configurations from the design and construction aspects. Their inclusion in guideway design should be carefully evaluated from the view point of aesthetics and economy.

A detailed examination should be made of constructibility of a steel guideway compared with concrete in specific locations along the typical alignment taking into consideration such factors as location of the guideway on the street, utility relocation, traffic disruption, access for construction vehicles, etc. This would bring to light a more realistic appreciation of the advantages and disadvantages of each system.

The designs of the candidate tangent and curved sections should be further refined to improve economy.

Hybridization of the typical alignment should be carefully evaluated. Specific lengths of guideway may favour either steel or concrete when viewed in terms of site specific parameters such as aesthetics, sound transmission, constructibility and economy. Suitable breaks on the alignment such as stations, long spans, the escarpment, etc. could be identified as change-over points from one material to the other.

Although considerable research has been done into the use of protective coatings on concrete which will reduce staining there seems to be a 'credibility gap' between what we know is available in the form of new acrylic and epoxy coatings from the manufacturers and what has actually been successfully employed in practice. This item should be researched in greater depth with the coatings industry.

The steel design should be verified for the effect of 5.5 million cycles of loading with the half full vehicle live load.

The principles to be adopted by a forthcoming MTC design manual for protection of reinforcing bars in concrete bridges against the effects of salt spray should be reviewed for their applicability to this guideway.

10. ITEMS EXCLUDED FROM THE STUDY

The following is a brief list of items excluded from the study due to time and monetary constraints. Their influence on the decision as to whether steel is a viable alternative to concrete for the guideway is generally small and inconsequential. They should, however, be examined, in a later phase of this Study, in terms of the same parameters.

They are:-

Cantilevered steel columns

The Station/Guideway interface

Foundation design for the steel guideway

Crossheads for 35 m radius curved dual guideway

Columns of other than circular cross-section

Tunnel design - steel or concrete liners

Cross-overs

Across-street bents

11. REFERENCES

1. The Weather Coat Process - Steel for Maintenance - Free Service
- Nippon Steel
2. Bridging with Steel - Technical Bulletin 19 - Stelco - July 1977.
3. Stelcoloy Weathering Steel - Technical Brochure - Stelco - March 1975.
4. CAN3-S6-M78 - CSA - Design of Highway Bridges
5. An Evaluation of Steel and Concrete Guideway Sections for I.C.T.S.
- Grouni, Billing and Prabudy
- MTC - April 1979
6. Trip Report - Visit to Modern North American Transit Systems - ABAM
Report
No. A80104-1.2-033 Nov. 1980.
7. Elevated Steel Guideway Concepts for use in ICTS - CSICC and Read
- Harris Engineering Nov. 1975
8. Atmospheric Corrosion of Metals - NRCC - CBD170-1975.
9. AGT Guideway and Station Technology - Guideway and Station Concepts
- USDT - UMTA July 1979.
10. Steel Structures for Mass Transit Systems-AISI.
11. Ontario Highway Bridge Design Code - MTC 1979.
12. Technical Design Proposal for Intermediate Capacity Transit
Demonstrative System Vol. III - Hawker Siddeley Canada Limited - Cole,
Sherman & Assoc. 1975
13. Estimation of the Natural Frequencies of Continuous Multispan Bridges
RR219 - MTC Jan. 1979.
14. Standard Specifications for Highway Bridges - The American Association
of State Highway and Transportation Officials (AASHTO).
15. W59-1977 - CSA - Welded Steel Construction
16. CAN3-A23.4-1978 - CSA - Precast Concrete - Materials and Construction
17. CSA Standard A251 - Qualification Code for Manufacturers of Architectural
and Structural Precast Concrete.

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.4

ACOUSTICAL ANALYSIS

WILSON, IHRIG & ASSOCIATES INC.,
OAKLAND, CALIFORNIA

MAY, 1981

TABLE OF CONTENTS

	<u>Page</u>
1.0 INTRODUCTION AND SUMMARY OF RESULTS	1
2.0 AERIAL GUIDEWAY STRUCTURAL CHARACTERISTICS	4
3.0 STRUCTURAL NOISE RADIATION	9
4.0 EFFECT OF STRUCTURE TYPE AND DAMPING	12
5.0 PROJECTED WAYSIDE NOISE COMPARISON	18
6.0 COMMUNITY NOISE IMPACT EVALUATIONS	21
FIGURES	25
REFERENCES	41

1.0 INTRODUCTION AND SUMMARY

A fixed guideway transportation system planned for the City of Hamilton, Ontario, is to be based on the Intermediate Capacity Transit System design (ICTS) developed by Metro Canada Ltd. The vehicle employed is driven by a linear induction motor along standard gauge track. For the proposed ICTS in Hamilton, much of the system will be on an aerial guideway. Compared with other transit vehicles in use today, the ICTS vehicles are considerably lighter (21,000 kg @ crush capacity) and operate at much slower speeds (72 km/hr cruise speed). Consequently the wayside noise from the ICTS system is, in general, significantly less than that associated with other transit systems.

The City of Hamilton has expressed a desire to use steel in construction of the aerial guideway as an alternative to an all-concrete structure. However, based on existing experience in the transit industry, the use of undamped steel plates as part of composite steel/concrete aerial structures can result in community noise levels due to train pass-bys that are potentially unacceptable. This report summarizes the findings of an analysis made to determine the comparative wayside noise levels of trains operating on three alternative, aerial guideway designs and the relative impact on the wayside community.

Bases on measured noise and vibration data taken by Metro Canada at the ICTS prototype test track in Kingston, Ontario, and a prediction model derived from a combination of empirical data and theoretical analysis, noise levels due to train pass-bys have been projected for the composite steel/concrete guideway with and without damping. These are compared with the measured data obtained at the prototype test track which has an all-concrete aerial guideway.

The noise impact analysis of the composite steel/concrete guideway is based upon the currently available data provided by Metro Canada from tests performed using the dual guideway concrete aerial structure at the test facility in Kingston, Ontario. As soon as more data become available, additional evaluation is advisable, since if the guideway and vehicle

system characteristics ultimately utilized for the Hamilton ICTS are significantly different from those used in this analysis, the results may be somewhat different.

The proposed Hamilton ICTS will be routed through three basically different areas of the city: Central Business District (CBD), Escarpment, and Mountain. Each of these areas has a unique existing noise environment and requirement that should be taken into consideration. An assessment of the impact of wayside noise on the community in the three areas was made using the predicted wayside noise determined by this analysis for the three alternative aerial guideways. To determine comparisons of the predicted wayside noise the resulting interior noise in typical construction buildings was determined for typical train pass-bys.

The general conclusions that can be drawn from this study are:

- (1) Even with the expected higher levels of structural noise radiation, the noise actually radiated from the wheel/rail interaction and the transit cars is expected to dominate the A-weighted wayside noise levels from normal operations on both the concrete and steel/concrete composite Hamilton ICTS structures, at least for outdoor noise.
- (2) With the use of constrained layer damping treatment applied to the steel plates, the Hamilton ICTS steel/concrete composite structure is expected to provide essentially the same performance regarding wayside noise as the all-concrete structure.
- (3) The outdoor A-weighted noise levels are not expected to differ significantly with either the concrete, undamped composite, or damped composite aerial guideway.
- (4) In the CBD, unless there should be an area with significant number of lightweight residential structures (as contrasted to high-rise masonry structures), there will not be any significant difference in noise impact from the three types

of aerial guideway.

- (5) In the Escarpment area the noise impact will also be essentially the same with any of the three types of aerial guideway, because this area is primarily open space. However, if there are any lightweight residential buildings located near the guideway in this area, the undamped steel/concrete composite aerial guideway would result in greater interior noise levels at low frequencies, and for such conditions damping should be used.
- (6) In the Mountain Region, either the all-concrete or damped (using constrained layer damping) steel/concrete composite aerial guideway would result in the same noise impact. However, the high levels of low frequency rumble noise from the undamped steel girder structure will result in a significantly increased impact inside buildings compared with the other two alternative guideways. Only in areas where there is a very high volume of nighttime, commercial traffic in existence would it be possible to use the undamped composite structure. The evaluation of those areas for which the steel girder requires damping should include consideration both of the first and second row of houses near the guideway structure.
- (7) A cost estimate using presently available data for the purchase and installation of constrained layer damping (not including the steel damping plate) was made. The estimated cost is \$170 (Canadian)/m. per single lane guideway beam. This assumes a 37% coverage of the steel plates in the guideway girder, a typical figure for such damping applications.

2.0 AERIAL GUIDEWAY CHARACTERISTICS

This section discusses characteristics of the aerial structure and track which are relevant to the prediction of wayside noise. Due to the similarities between the ICTS aerial guideway and the aerial structure used in the San Francisco Bay Area Rapid Transit system (BART), prediction models and measurements made at BART have been used as a basis for the ICTS model. In addition, a prediction model developed for the Metropolitan Atlanta Rapid Transit system (MARTA) and for the Washington Metropolitan Area Transit Authority system (WMATA) and based in part on the prediction models and data obtained from BART has been used.

Figures 1 and 2 show the basic cross-section configuration of the ICTS aerial guideway in both the all-concrete and composite form. Figures 3 and 4 show the corresponding cross-sections of the BART aerial structures. The steel/concrete section from BART is of the Walnut Creek Bridge for which noise and vibration measurements have been made (Reference 11) for comparison with similar measurements at the all-concrete BART aerial structure.

The ICTS concrete aerial guideway consists of standard gauge track (56 kg/m) which is resiliently mounted with Hixson rail fasteners to a concrete deck. The concrete deck in conjunction with a trapezoidal concrete box underneath forms the rail support girder between support columns. In the case of dual lane track, each track is supported by its own girder structure and the two guideways are connected only at support piers.

The ICTS design incorporates parapet walls on both sides of the guideway which will reduce radiation of undercar noise (eg., wheel/rail and motor noise). In contrast with other transit system aerial structures, the ICTS parapet walls are an integral part of the deck structure. In other transit systems (eg., BART, WMATA, MARTA) instead of a parapet, sound barrier walls are employed. Although the sound barrier walls are rigidly attached to the deck they are not structural members and therefore, do not appreciably affect the stiffness of the girder.

By making the walls integral to the structure, as in the ICTS design, the forces from the rail supports interact with a higher impedance and consequently the vibration levels of the guideway and, ultimately, radiated noise are lower. Any possible increase in vibration levels of the parapet walls due to their being integral parts of the structure should be minor. Furthermore, the importance of sound barrier walls in reducing radiated vehicle noise far overshadows any increased radiation effects.

The BART aerial structure, apart from differences in dimensions and the parapet walls, is basically very similar to that of the ICTS. The major difference between the two systems is, of course, in the transit vehicles and the speeds at which they travel. It is assumed in this analysis that the relationship with regard to vibration level differences between all-concrete and steel/concrete composite structures does not depend significantly on the transit vehicle's characteristics or its speed within the range of parameters covered by the various systems considered in this analysis. More explicitly, the vehicle-to-structure weight ratio varies considerably among BART, WMATA, and MARTA, and the prediction analysis used for WMATA and MARTA indicated levels that were very close to measured levels obtained after operations began.

Several different span lengths will be used in construction of the guideway for the Hamilton system. Consequently, in the case of the steel/concrete composite girder the dimensions of the steel box will vary depending on the span length. The all-concrete guideway typically spans 30 m. and will have the same cross-section dimensions for spans longer or shorter than 30 m. For the purpose of this study it was considered sufficient to compare the worst case steel/concrete configuration with that of the all-concrete. As discussed later the geometry for the 24 m. span provides the worst case from the standpoint of lower mechanical impedance and also from the standpoint of resonant modes.

There are three main design features which are required in order to minimize or reduce the noise generated by an aerial structure:

- (1) Resilient rail fastenings to reduce the magnitude of vibration forces transmitted from the rail to the aerial structure,
- (2) Heavy or massive decks to reduce the magnitude of mechanical vibration amplitude in response to the vibratory forces which result from operation of transit trains over the rails (i.e., to increase the mechanical impedance of the structure), and
- (3) Damping to minimize the amplification of mechanical vibration caused by resonances of the aerial structure components.

The ICTS already incorporates items (1) and (2) into its design and item (3) concerning damping is a major purpose of this study.

An all-concrete aerial structure provides, by the nature of the structure material, both a massive deck to provide inertial resistance to motion induced by the vibratory forces from the transit cars and a relatively high degree of internal damping due to the nature of the material. Usually the mass of concrete necessary to provide adequate structural strength and rigidity provides sufficient inertial mass to limit the amplitudes of vibration such that with resilient rail fasteners the A-weighted noise levels generated by the structural vibration are less than the A-weighted levels produced directly by the car.

Measurements of mechanical vibration levels on the BART aerial structures and measurements of the wayside and undercar noise for various operating conditions, including the use of sound barrier walls, demonstrate that the noise from the aerial structure vibration is less than the noise produced by the car except at very low frequencies where the two are comparable.

Although the vehicle characteristics of the ICTS are significantly different from those of the BART cars, and the vibration and noise data available for the ICTS are not as extensive as that for BART, it appears that similar conclusion can be reached regarding the ICTS. The conclusion is that, except at low frequencies, the noise radiated from the all-concrete aerial guideway vibration is less than the noise produced by the vehicle itself.

Table I gives a summary of some of the physical parameters and measured values used to compare the various aerial structures.

Since these data were calculated at the mid-span of the structures and ignore any increased weight due to cross braces, diaphragms, rail pads, rails, or the increase in cross-section area near the ends, they do not represent the exact overall stiffness and mass of the various structures. However, since the data are to be used only to approximately compare the various structures, more exact calculations of mass and stiffness are not necessary. The comparisons are valid as long as the estimates of stiffness and mass for the various structures are done in a consistent manner.

TABLE I GENERAL PARAMETERS OF THE BART AND ICTS AERIAL STRUCTURES

Reference Figure	Structure Type	Total Weight* (N/m)	Width (m)	Stiffness (N/m)	Low Frequency Input Impedance (N-sec/m)
4	BART - Concrete (21 m span)	3.9×10^4	3.5	7.4×10^9	$6.7 \times 10^3 \sqrt{f}$
5	BART - Undamped Composite (Walnut Creek Bridge) (26 m span)	3.0×10^4	4.0	8.7×10^9	$3.8 \times 10^3 \sqrt{f}$
1	Hamilton ICTS - Concrete (24 m span)	3.0×10^4	2.8	10.2×10^9	$6.0 \times 10^3 \sqrt{f}$
2	Hamilton ICTS - Damped Steel/concrete Composite (24 m span)	1.9×10^4	2.8	7.1×10^9	$4.1 \times 10^3 \sqrt{f}$
2	Hamilton ICTS - Undamped Steel/concrete Composite (24 m span)	1.9×10^4	2.8	7.1×10^9	$4.1 \times 10^3 \sqrt{f}$

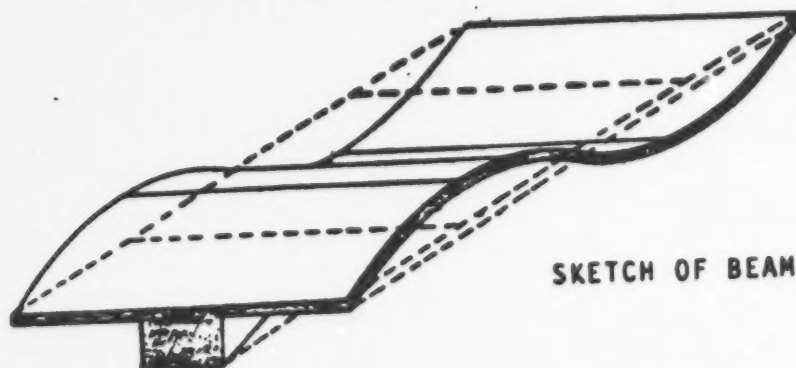
*These weights are at the mid-span and do not include the rails or rail support pads

3.0 STRUCTURAL NOISE RADIATION

Throughout this discussion it is assumed that resilient direct fixation rail fasteners will be used on the aerial structure for the Hamilton system, and all of the estimated noise levels presented assume use of resilient rail fasteners. It is further assumed that the wheels and rails will be well maintained to provide smooth uniform operating surfaces. The effect of rail grinding in reducing wayside noise was shown in a series of tests at the BART system facilities (8). The tests were done with the car operating on new smooth wheels and show that grinding of the rail produced a substantial noise reduction over most of the frequency range.

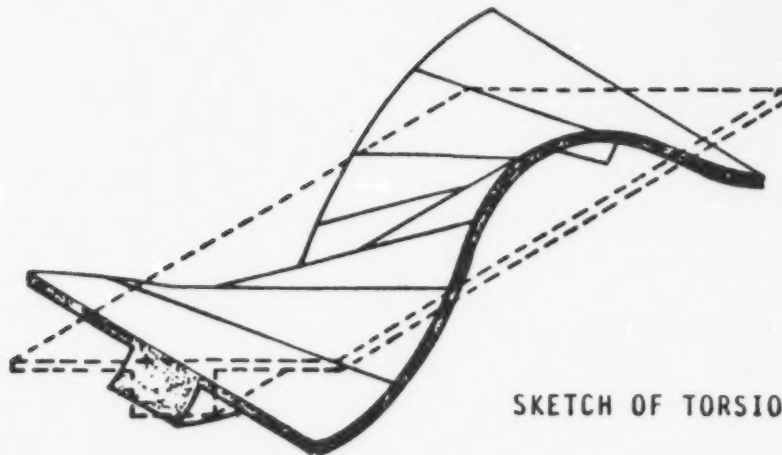
In order to estimate the vibration levels of the aerial structures and to compare the noise radiation of the various types of aerial structures, it is first important to understand the types of vibratory motion that can occur in the structure. Some of the types of motion that are expected to occur and that may cause significant noise radiation are:

- (1) **Beam Bending:** In this type of motion, the entire structure acts as a single large beam with the entire cross-section moving in phase. In this case the aerial structure between two piers can simply be modeled as one large, long beam. This is probably the primary noise radiation motion experienced with the BART concrete aerial structure.



SKETCH OF BEAM MODE

- (2) **Torsional Motion:** For torsion, the structure is once again modeled as one large, long beam, except this time the load causes a twisting of the structure. As the wave motion propagates down the aerial structure, each cross-section rotates about its center of twist without experiencing much distortion.



SKETCH OF TORSION MODE

- (3) Plate Bending: In this type of motion, bending occurs in two directions, instead of just one like a beam. Plate motion can be important in both the steel plates of composite steel/concrete structures and where a large concrete deck is used such as in MARTA structures. The steel plates will vibrate in plate modes at a much lower frequency than the concrete part of the structure and will cause a large increase in the noise radiation in some octave bands.

There are other types of vibratory motion that can occur on aerial structures, but the three discussed above are the most significant in terms of low frequency noise radiation. Because the loading of the ICTS deck by the transit vehicle is essentially symmetric in terms of a short-time average, as opposed to the MARTA deck which is eccentrically loaded, it can be assumed that the excitation of torsional motion is insignificant compared with the other two types of motion. Consequently the prediction model only includes the effect of beam bending and plate bending.

As previously noted, it is assumed in this analysis that the level of response of the all-concrete structure relative to that of the steel/concrete composite is independent of the vehicle characteristics and its speed. This assumption is necessary due to the lack of data for light-rail vehicles on steel/concrete composite structures. Based on the good agreement between the predicted wayside noise levels and the actual measured values at MARTA and WMATA this assumption is appropriate.

Noise and vibration data for the ICTS system was obtained by measurement of train pass-bys on the Kingston Test Track (14). Figure 5 indicates the location of the microphones. Data were obtained for a single vehicle operating at three constant speeds (36, 54, and 72 km/hr). The baseline vibration and noise data is indicated in Figures 6 and 7. For the purpose of comparison only, measurements were also made with the ICTS vehicle accelerating past the microphones at various rates. However, the latter set of data were not included in the noise prediction models.

Following is a summary of the general methodology used to predict the expected noise levels radiated from the three alternative ICTS elevated guideway designs.

- (1) Determine contribution to total wayside noise of noise from car and noise from structure for all-concrete elevated guideway.
- (2) Apply model analysis to predict structure-radiated noise for composite steel/concrete for damped and undamped configuration.
- (3) From information obtained in (1) and (2) above, predict combined wayside noise from vehicle and steel/concrete composite structure for the various vehicle speeds mentioned.
- (4) Using the predicted noise levels obtained in (3) for steel/concrete composite structures and the measured noise level for all-concrete for the case of a single vehicle pass-by, predict the wayside noise level for a 4-car train pass-by for the three alternative ICTS structural designs.

4.0 EFFECT OF STRUCTURE TYPE AND DAMPING

It has been shown in analysis of the BART (5) and (11), WMATA (10), and MARTA (13) aerial structures that the differences in wayside noise levels resulting from train operations on the three types of aerial structure configurations considered herein - all-concrete, damped composite steel/concrete, and undamped composite steel/concrete - occur in the frequency range below 200 Hz. These analyses show that the low frequency rumbling noise is expected to be very significant and a potential source of annoyance from the undamped composite structure, but that provision of adequate structural damping will reduce the rumble of the composite structure to a level slightly above that of the all-concrete structure.

It has also been observed that the importance of rumble is greatly increased for outdoor sounds that will be perceived inside buildings, since low frequencies are attenuated much less than high frequencies in penetrating a typical building. Lightweight residential buildings are especially poor in attenuating low frequency sound.

Although there is a considerable amount of data available on the effectiveness of various types of damping treatments, information on the techniques and results of damping vibrations of bridge and girders is limited. However, some data and observation of damping treatment of rail transit structures have evolved from research and experience with the BART, WMATA, and MARTA systems.

The potential desirability of using steel beam composite steel/concrete girders led to the performance of some systematic studies of noise radiation from aerial structure girders by Stanford Research Institute (SRI) early in the BART Test Track program. The objective was to determine if a composite steel/concrete girder would perform as well as an all-concrete girder (1-4).

Later tests performed by Wilson, Ihrig and Associates, Inc. (WIA) on the BART Test Track measured both girder and rail vibration levels along with radiated noise levels using a number of different types of track fasteners,

several wheel types, and smooth ground and unground rail (5-8). These tests were all carried out on all-concrete aerial structures. Following the beginning of revenue service on the BART system another study was performed by WIA on the sound radiating characteristics of the composite steel/concrete Walnut Creek Bridge and other BART composite structures including one with surface type damping applied (11). This study produced information on the noise levels radiated from various types of aerial structures on the BART system during normal revenue service.

The basic damping techniques which can be considered are surface application of damping mastics or compounds, application of a sheet of viscoelastic polymer damping material followed by a "constraining layer" of steel, or incorporation of constrained layer damping into the plates used in fabricating the aerial structure. The last two options are quite similar in that they utilize a layer of viscoelastic damping material between two stiff metal plates which maximized the effectiveness of a given amount of damping material. The distinction between the last two methods is that surface application of constrained layer damping adds dead weight to the structure because the constraining layer is not carrying structural loads, whereas the steel plates on both sides of the constrained layer of viscoelastic damping material can carry structural loads in a properly engineered system fabricated with damped, sandwich plates.

The use of an unconstrained, surface application of damping mastics or compounds is relatively impractical for plates of the thickness employed in aerial structures, since the thickness of such treatments must be large (up to several times the thickness of the plate being damped) to achieve the large amount of damping required to suppress aerial structure rumble. As a result, the required weight of unconstrained damping treatment can reach 50% of the weight of the panel being damped.

Attainment of adequate adhesion of unconstrained layers can also be a problem and, in fact, unconstrained, water-based damping material applied to one composite bridge of the BART system had to be removed and re-applied due to inadequate adhesion.

Experience at BART indicates that unconstrained damping treatments do not give satisfactory attenuation of those low frequency vibrations producing rumble. A research project undertaken by Stanford Research Institute (3), in which a 28 m mockup of a BART composite aerial structure was treated with mastic damping compound up to the thickness of the plates (16 mm) in some areas and the noise produced by vibrating the bridge was measured, showed that reduction of low frequency vibration by the damping treatment was slight.

The SRI measurements did show that the interior damping of the girder was effective in reducing the overall noise, however, it was also found that the substantial reduction of the noise was in the frequency region from 300 to 6000 Hz. In some cases the interior damping produced little or no reduction of low frequency noise and in one case it was found that the partial damping treatment tested actually increased the noise level in the region near 60 Hz. Full damping treatment did result in a minor reduction of the low frequency vibration and noise.

Since the high frequency noise radiated by the aerial structures is lower than that radiated by the wheel/rail system, even with a sound barrier wall, the reduction seen in structure radiated noise at 300 to 6000 Hz is of negligible benefit.

Noise measurements have also been performed at the Marin Avenue Bridge of BART, a composite steel/concrete aerial structure on the interior surface of which water-based mastic damping compound was applied. The results of the measurements at Marin Avenue confirm the results reported by SRI on the tests with the damped American Bridge structure tested in 1964. That is, that the interior surface applied damping actually results in only a small degree of reduction of low frequency vibration.

A damping treatment which can be "tuned" to the frequency range of concern (about 20 Hz to 200 Hz), and which can achieve adequate motion damping to be fairly sure of reducing the vibration up to 8dB, is the constrained layer damping "sandwich" or "layered" concept. Therefore, one possible way to achieve satisfactory results is to apply a constrained

layer damping treatment to the side and bottom girder plates with the design optimized for giving maximum damping in the region of 20 to 200 Hz.

The constrained layer damping concept maximized the effective damping provided by a given amount of viscoelastic damping material by creating high shear strain across the constrained layer in areas of maximum flexure. Typical treatments for the aerial structure plates employ a viscoelastic layer from 1 to 5 mm in thickness and a constraining layer of steel about 50% of the thickness of the plate being damped. A typical layout is shown in Figures 8 and 9.

A factor influencing the design of the damping is the distribution of damping over the area of the plate. Since the energy dissipation is maximized at areas of maximum flexure and is not necessarily very great at areas of maximum displacement of the plate (unless that area corresponds to a high flexure point for another mode of vibration), the total amount of treatment and therefore its total weight and cost can be reduced by limiting treatment to the areas of the plate where it will be most effective. The optimum locations and areas of treatment for the modes of vibration contributing to the rumble can be determined through use of a finite element computer program which can be used during final design.

The methods used for calculating the design of constrained layer damping and predicting the effectiveness of a design were originally developed by Bolt Beranek and Newman and subsequently improved by scientists at E. I. DuPont. A detailed calculation has not been performed for the ICTS elevated guideway, but by extrapolation from a damping system designed for the MARTA system, the following rough estimate of non-structural, constrained layer treatment for the ICTS elevated guideway can be made. The weight of the damping treatment will probably be approximately 60 kg/m, which is about 3% of the weight of the undamped composite structure. Note that tradeoffs are possible between constraining layer thickness, polymer chemistry and thickness and damping effectiveness. Also, the optimization of damping placement and/or incorporation of the constraining layer into the structural design of

the girder can significantly reduce the added dead weight of the damping treatment.

The results contained in this report assume that the girders which are part of the composite structure are symmetrical tangent beams. These results should not be applied to curves or to special trackwork sections.

Further evaluation is necessary to determine the wayside noise from special trackwork on the steel/concrete composite structure, since among other factors affecting noise, a moveable switch frog is currently under consideration. For the curved sections of the composite structure, if in fact the girder is curved, the structure resonances will be suppressed without damping with the resulting wayside noise impact less than that of the undamped tangent structure.

Some of the general conclusions that can be derived from the reports of measurements on the BART system (1-7 and 11) are:

- (1) The application of damping material to the steel webs of composite steel/concrete aerial structures reduces girder plate vibration by 4 to 12 dB in general.
- (2) Application of mastic type damping material to the steel plates of composite steel/concrete girders reduces the structure-radiated noise level by 5 to 10 dB (2 and 3), although most of the reduction is for the frequency range above about 250 Hz with little reduction of low frequency noise.
- (3) Application of constrained layer damping material to the steel plates of the composite steel/concrete girders reduces the structure-radiated noise level from 3 to 10 dB in the low frequency range if properly designed.
- (4) The net result is that a damped composite steel/concrete aerial structure girder produces about the same or slightly greater A-weighted noise

levels than an all-concrete girder. An undamped steel/concrete composite structure radiates approximately the same middle and high frequency noise as an all-concrete structure, however, the low frequency noise radiated from an undamped composite structure is approximately 10 to 14 dB greater than radiated from an all-concrete structure.

5.0 PROJECTED WAYSIDE NOISE COMPARISON

The results of the analysis leading to estimation of the noise levels due to train operations on the proposed Hamilton ICTS elevated guideway are summarized in Figures 10 through 14. Figure 10 indicates the predicted contribution of structurally radiated noise to the total wayside noise due to operation of a single ICTS vehicle at 72 km/hr on an all-concrete aerial guideway. Figures 11 and 12 compare the wayside noise due to operation at 54 km/hr and 72 km/hr respectively, of a single ICTS vehicle on the three alternative aerial guideway structures. A similar comparison is made in Figures 13 and 14 for a 4-car train. Table II summarizes the estimated A-weighted (dBA) wayside noise levels. The data in Table II show that the outdoor A-weighted noise level will be dominated in all cases by the noise produced by the car itself. Thus the perceived outdoor noise level will be approximately the same, regardless of the type of elevated guideway structure. However, the indoor noise level, in the case of adjacent lightweight construction, residential buildings, will be affected by the type of elevated guideway structure on which ICTS vehicles operate (see section on "Community Noise Impact Evaluation").

TABLE II SUMMARY OF ESTIMATED NOISE LEVELS
15 m FROM NEAR TRACK CENTERLINE -
WITH POLISHED RAIL

<u>Structure</u>	Total Wayside Noise (in dBA)			
	<u>54 km/hr</u>		<u>72 km/hr</u>	
	<u>Single Vehicle</u>	<u>4-Car Train</u>	<u>Single Vehicle</u>	<u>4-Car Train</u>
Hamilton ICTS - Concrete	63	67	67	71
Hamilton ICTS - Damped Composite	63	67	67	71
Hamilton ICTS - Undamped Composite	63	67	67	71

To summarize the most important results of the analysis:

- (1) The wayside A-weighted noise levels from vehicle operations on the Hamilton ICTS damped composite steel/concrete and all-concrete elevated guideway structures are expected to be approximately the same. The character of the noise makes these structures suitable for use in noise sensitive areas.
- (2) The high intensity, low frequency noise that will radiated from an undamped composite steel/concrete structure make this configuration unsuitable for use in noise sensitive or average-to-quiet residential areas. This is due to the large amount of low frequency noise that would be transmitted to the inside of adjacent structures.

Although there will not be significant differences between the two types of Hamilton ICTS structures in terms of A-weighted wayside noise levels, the frequency analysis of the wayside noise indicates some very significant differences. With the constrained layer damping treatment the noise radiated from the composite aerial structure will be essentially the same as that from the all-concrete structure. The primary difference will be 1 to 7 dB higher noise levels in the frequency range below 250 Hz.

Above 250 Hz, the radiation from the Hamilton ICTS undamped aerial structure is substantially the same as from the Hamilton ICTS concrete and damped composite structures. However, in the lower octave bands the levels radiated by the undamped composite structure will be greater. These greater levels in the low frequency region make the undamped composite steel/concrete structure unsuitable for use in average-to-quiet residential or other noise sensitive areas.

Due to the differences between the BART, MARTA, and WMATA structures and vehicles, for which noise data are available, and the Hamilton ICTS structure and vehicles some amount of uncertainty exists in the predicted noise levels. There will always be a certain degree of uncertainty in

calculations such as those done in preparation of this report. Unfortunately, it is not possible to mathematically derive a quantity such as a confidence interval for the estimated values of noise radiation. Intuitive estimates based on past experience are the best available at this time. We estimate that the predicted noise levels are accurate to within ± 2 dBA for the Hamilton ICTS concrete and damped composite structures, and about ± 3 dBA for the undamped composite structure. However, this uncertainty can and has been taken into account when the community noise impact evaluation was made, and is reflected in the conclusions.

6.0 COMMUNITY NOISE IMPACT EVALUATION

One of the most important aspects of this analysis is to determine the difference if any in wayside noise impact for the ICTS trains travelling on the three different aerial guideways. The present study shows that the major difference in the wayside noise is the large increase in low-frequency or rumbling noise when the vehicles operate on the composite steel/concrete aerial structure without damping when compared with the vehicles operating on the all-concrete structure (see Figures 11 - 14). The difference in the character of the noise from trains operating on the undamped composite structure when compared to trains operating on the all-concrete structure can be discerned by people outdoors specifically listening to a train passing by. However, for the typical person located near the guideway, the difference would not be noticeable. In any case, the noise for either type of girder will be of an acceptable level and will be on the order of 1- dBA lower than for a bus operating at the same speed.

The potential for impact from wayside noise with the vehicles operating on an undamped composite structure is inside nearby residential buildings, particularly during the nighttime hours when the background noise is lower resulting in an increase in sensitivity to noise.

Inside typical residences adjacent to the ICTS guideway at a distance of 15 m, a 4-car train travelling at 72 km/hr on the all-concrete or damped composite structure would produce low frequency noise (i.e. at the 31.5 Hz octave band) in the range of 50 to 60 dB which is acceptable. In terms of Noise Criterion Levels (a scheme which accounts for the low frequency sound and extensively used when specifying a desirable noise level inside an occupied space) these trains would produce a level of NC-25 or less inside these residences, which is also acceptable. For the same conditions, a train operating on the undamped composite structure would produce low frequency interior noise of 65 to 70 dB and a noise criterion level of NC-30 to -35.

The range for both of these measures indicate that the train passby noise would be noticeable to most people and could result in dissatisfaction and complaints from the affected residents.

The train passbys on the undamped composite structure would result in a noticeable rumble inside the nearby residences, although the actual noise level would be no greater than the street traffic for those residences facing a high traffic volume street. Thus, the potential for wayside noise impact due to this rumbling noise exists inside residences for those locations where houses will be adjacent to the ICTS undamped composite aerial guideway.

For those residences adjacent to the ICTS damped composite or all-concrete aerial guideway, the wayside noise levels inside the buildings will be acceptable and no noise impact from ICTS operations is expected.

The following is a discussion of the relative noise impact from trains operating on the different aerial guideways in three general community areas of Hamilton where the ICTS is proposed to operate; the Central Business District, Escarpment and Mountain.

For the Central Business District, trains operating on any of the three types of elevated structure should be acceptable and there will be essentially no difference in the noise impact or noise exposure for the steel guideway, either damped or undamped, and the concrete guideway. The only potential for difference in the noise impact of a steel girder guideway is from the high level of low frequency noise from trains operating on an undamped composite structure. This noise can be transmitted into the buildings adjacent to the alignment. However, in the CBD there is very little nighttime occupancy and the buildings tend to be of relatively heavy masonry construction. Further, in such CBD buildings one of the major noise sources is the heating and ventilation system for the building which tends to create background noise which has relatively high levels of low frequency noise. All of these factors tend to minimize any effect of the higher levels of low frequency noise from the undamped composite structure. Thus, unless there should be an area with

a significant number of lightweight residential structures (as contrasted to high-rise masonry structures) in the CBD, there will not be any significant difference in noise impact from the three types of aerial guideway. Further, the noise from any of the proposed aerial guideway assemblies should be acceptable, creating no significant noise impact.

The Escarpment Area is primarily open or park-like space or roadway corridor space. Thus, because any of the proposed aerial guideways will produce approximately the same outdoor noise levels and essentially the same noise impact for the levels associated with operations of the ICTS system on aerial guideway, the noise levels should be acceptable and, therefore, as previously discussed, there will be no adverse or significant noise impact from train operations on any of the three types of guideway in the Escarpment Area. Only if there should be an area where relatively lightweight residential type buildings were located near the guideway in the Escarpment, the potential for noise impact would be significant.

In the Mountain Region, where there are significant number of residences with associated nighttime occupancy, there is a difference between the potential noise impact for the concrete guideway or damped steel girder composite guideway and the undamped steel girder composite guideway. The small difference in low frequency noise level for the damped girder composite structure and the concrete structure is not sufficient to cause a difference in noise impact from train operations for those two types of girders. However, the high levels of low frequency rumbling noise from the undamped steel girder structure can create a difference in noise impact compared to the other two types of structures.

For sections of the alignment passing through commercial areas in the Mountain Region or along residential streets with very high traffic volume where the noise from traffic includes trucks and continues, with trucks, throughout the nighttime period, then the high levels of low frequency noise from the undamped composite structure will not create a significant noise impact compared to the concrete or damped composite structure. However, for areas where the alignment is located adjacent

to residences in a typical residential area where the traffic noise at night may be diminished and not have a large volume of trucks mixed in the traffic, or at the second row of houses from the busy street, there is likely to be a significant difference in the noise impact of the undamped composite structure compared to the all-concrete or damped composite structure. In such locations it is recommended that for the composite structure only the damped configuration be used to avoid this potential noise impact inside buildings. The evaluation of those areas for which steel girder requires damping should include consideration both of the first and second rows of houses near the guideway structure.

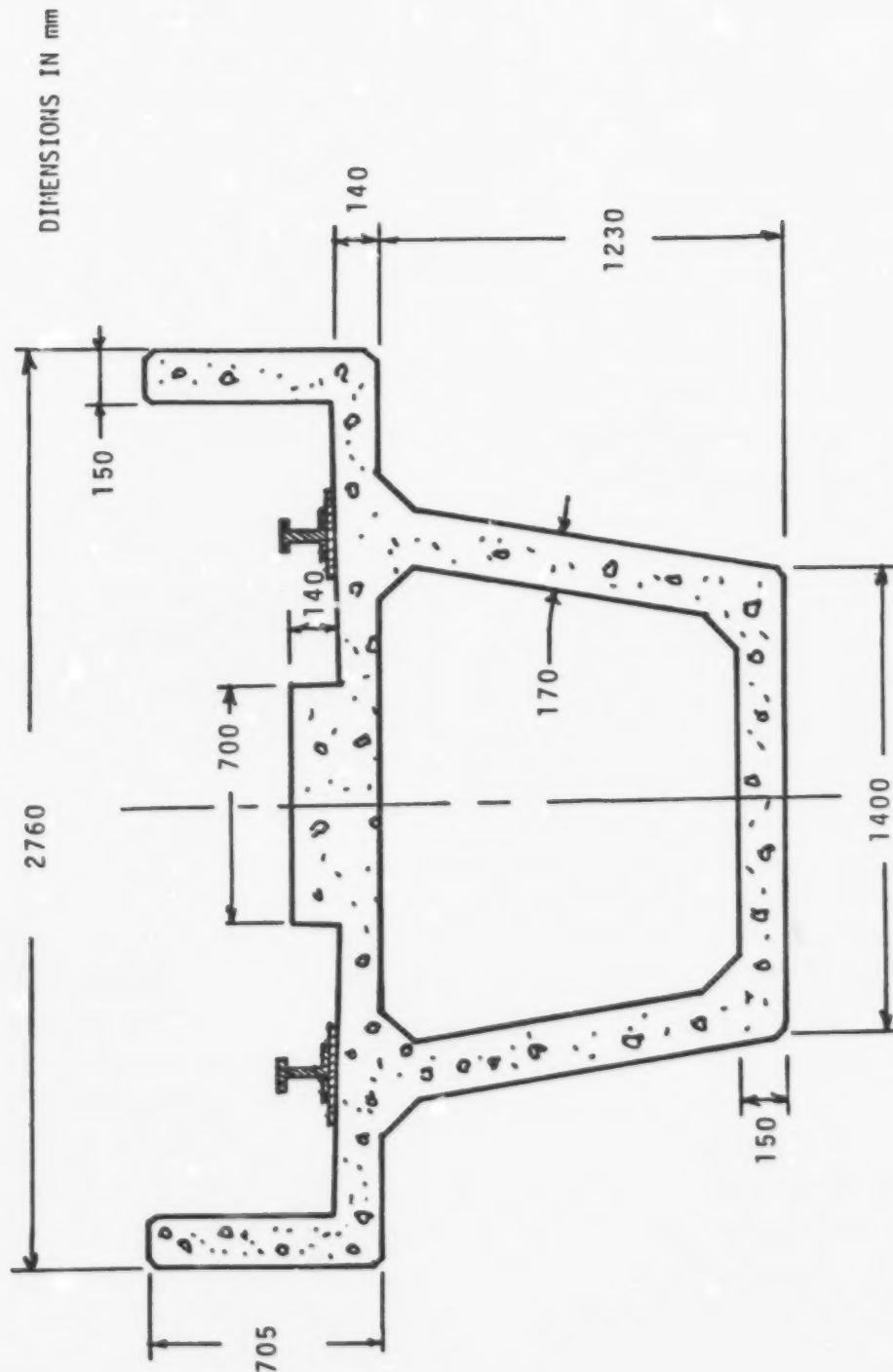


FIGURE 1 ICTS PRE-STRESSED CONCRETE AERIAL GUIDEWAY AT MIDSPAN

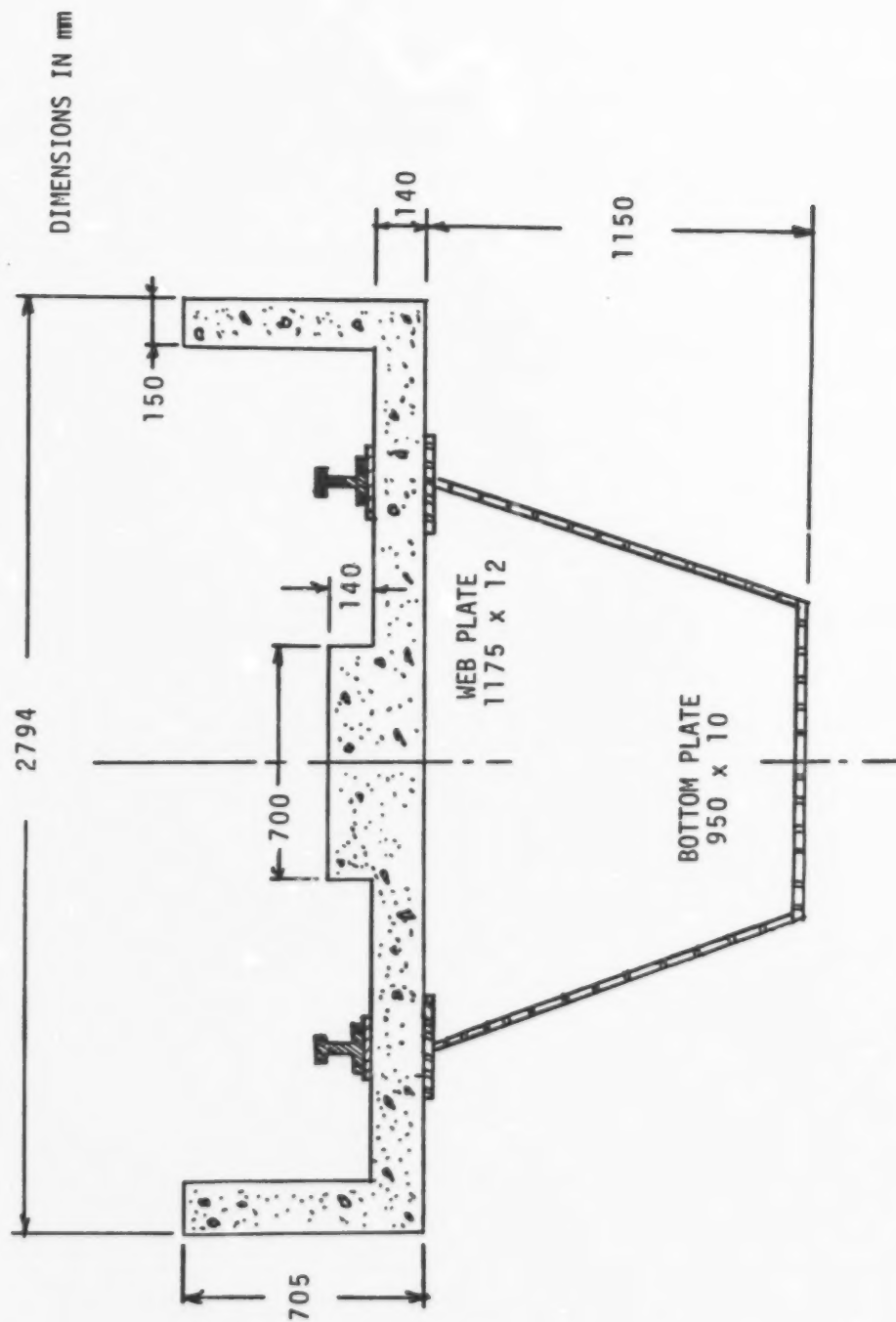


FIGURE 2 ICTS COMPOSITE STEEL/CONCRETE AERIAL GUIDEWAY FOR 24 m
SPAN - AT MIDSPAN

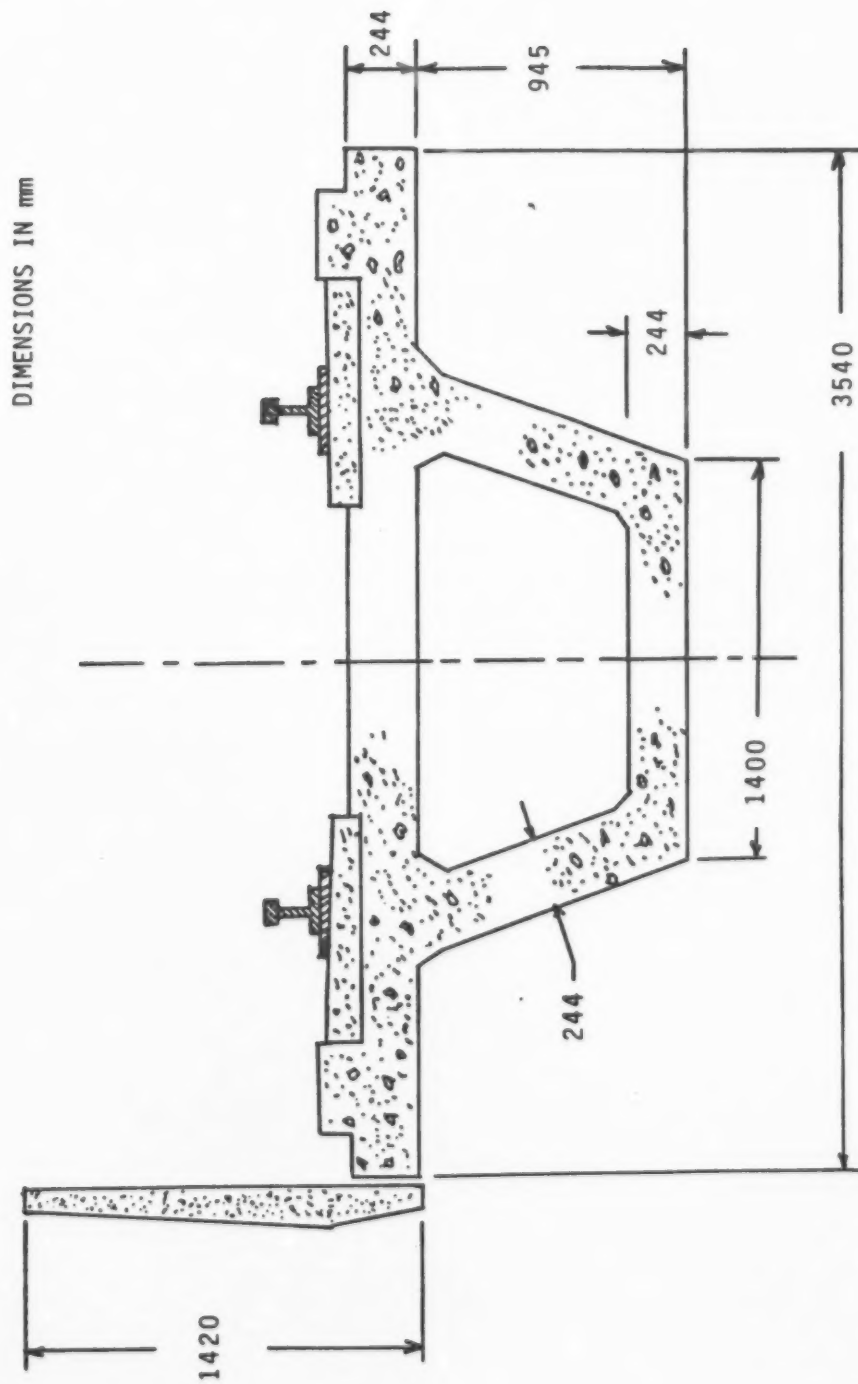


FIGURE 3 BART PRE-STRESSED CONCRETE GIRDER AERIAL STRUCTURE WITH A SOUND BARRIER WALL

DIMENSIONS IN mm

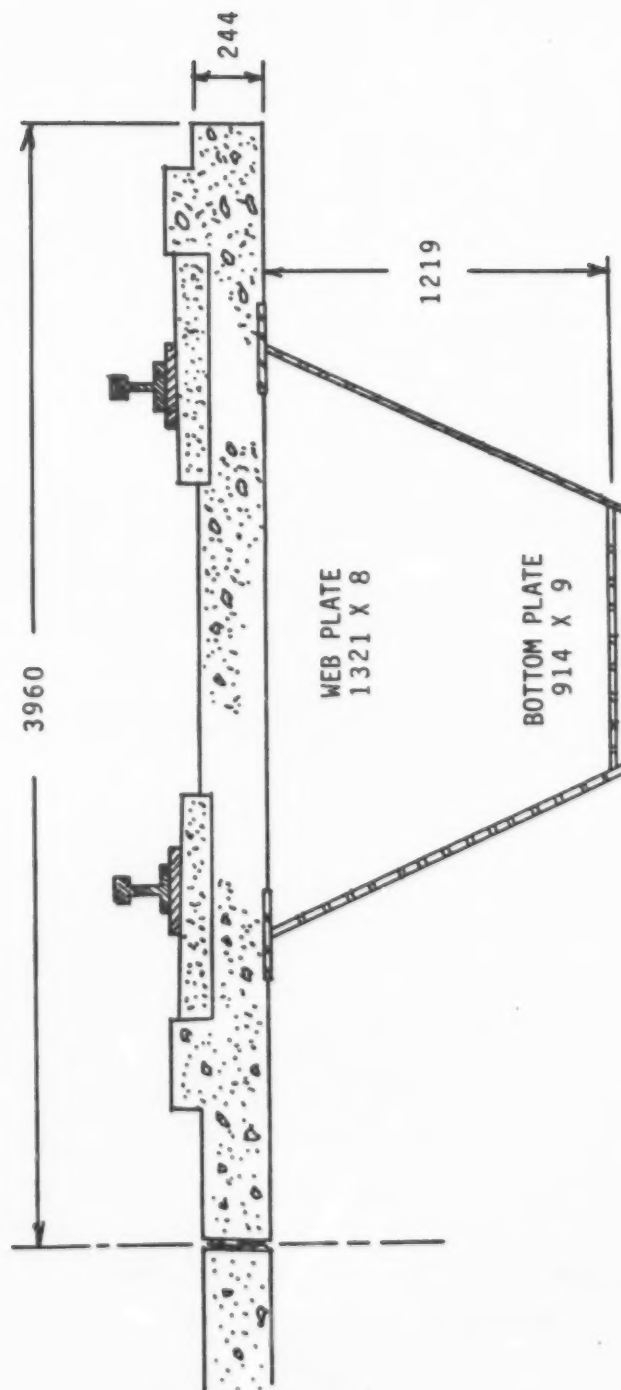


FIGURE 4 BART COMPOSITE STEEL/CONCRETE AERIAL STRUCTURE - WALNUT CREEK BRIDGE

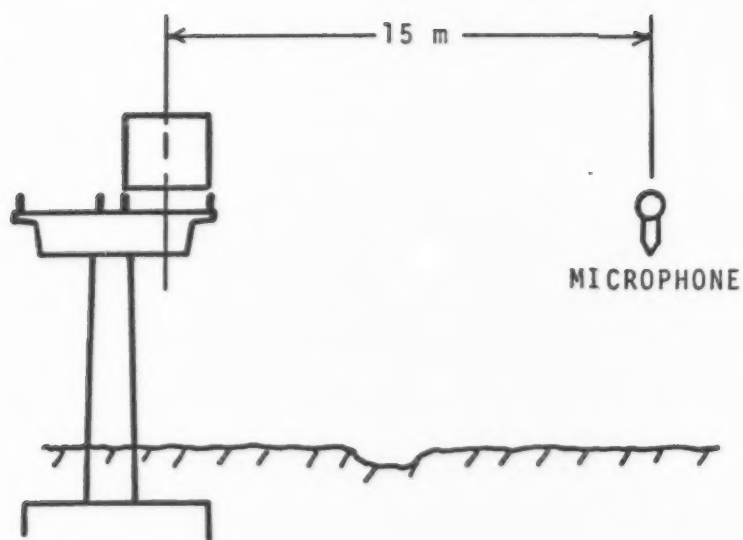


FIGURE 5 CROSS-SECTION OF AERIAL GUIDEWAY AREA
SHOWING MICROPHONE LOCATION

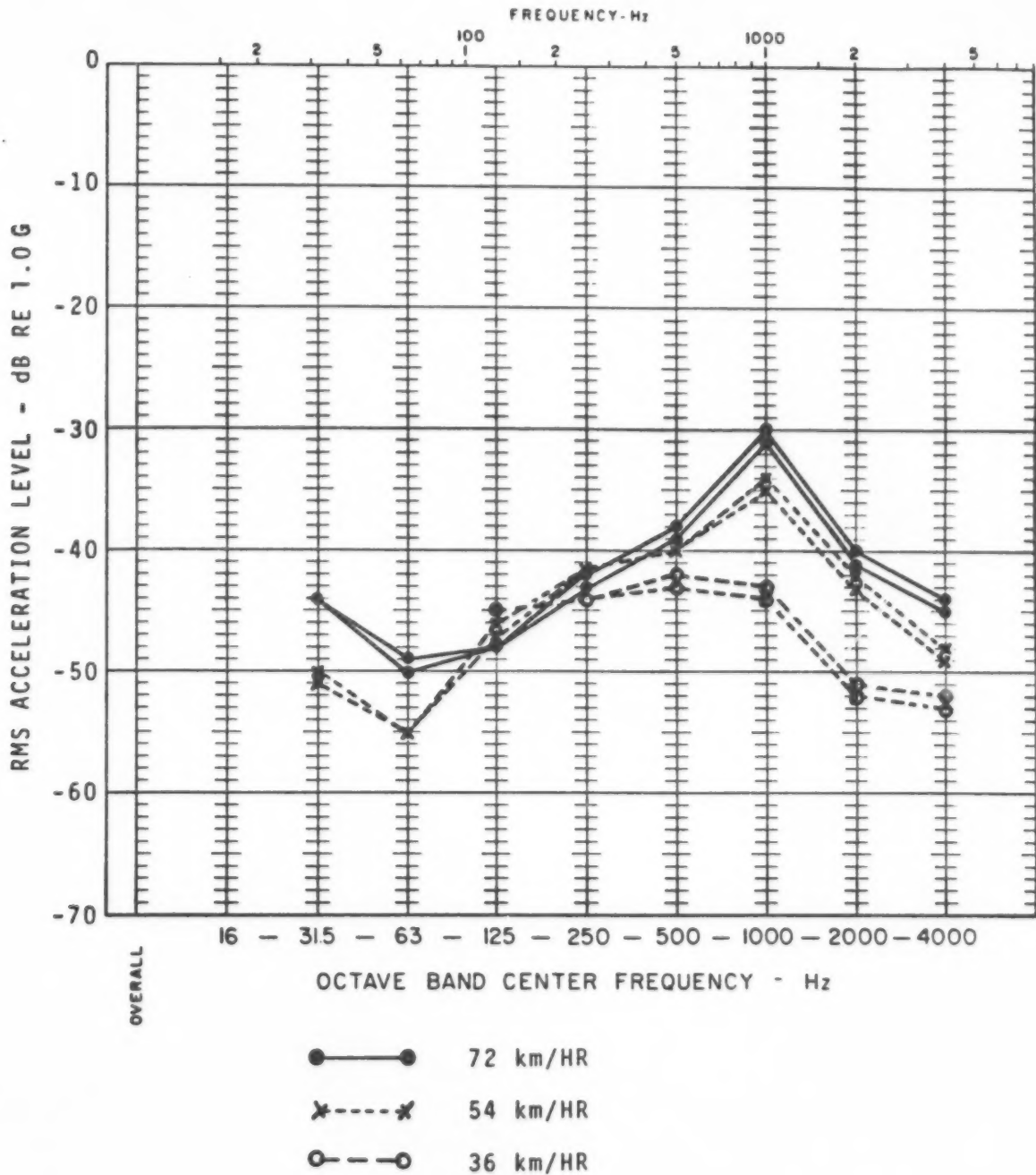


FIGURE 6 MEASURED DECK VIBRATION ON ALL-CONCRETE AERIAL GUIDEWAY - SINGLE VEHICLE PASSBY

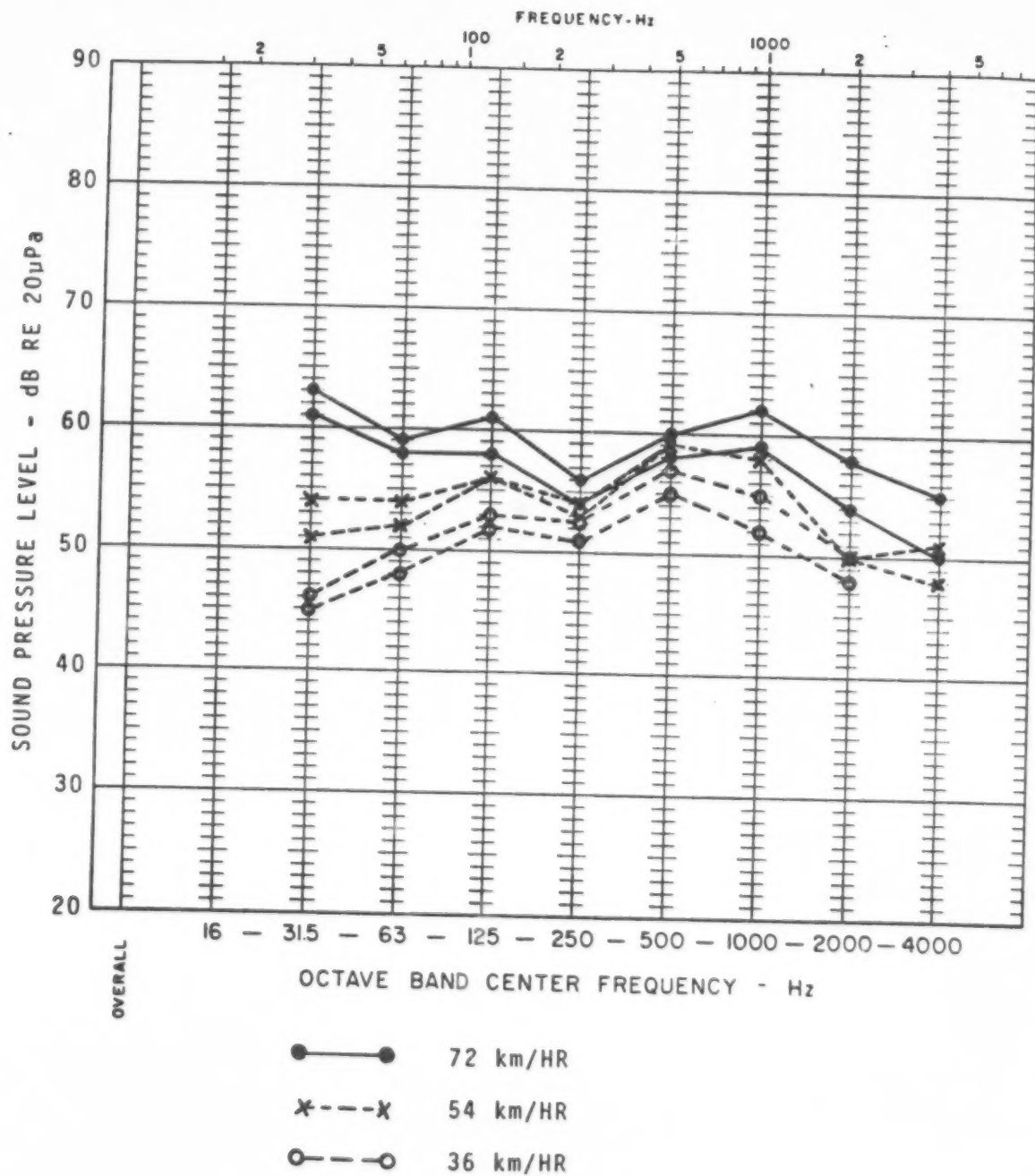


FIGURE 7 MEASURED WAYSIDE NOISE AT GUIDEWAY HEIGHT 15 m
FROM ALL-CONCRETE AERIAL GUIDEWAY - SINGLE VEHICLE
PASSBY

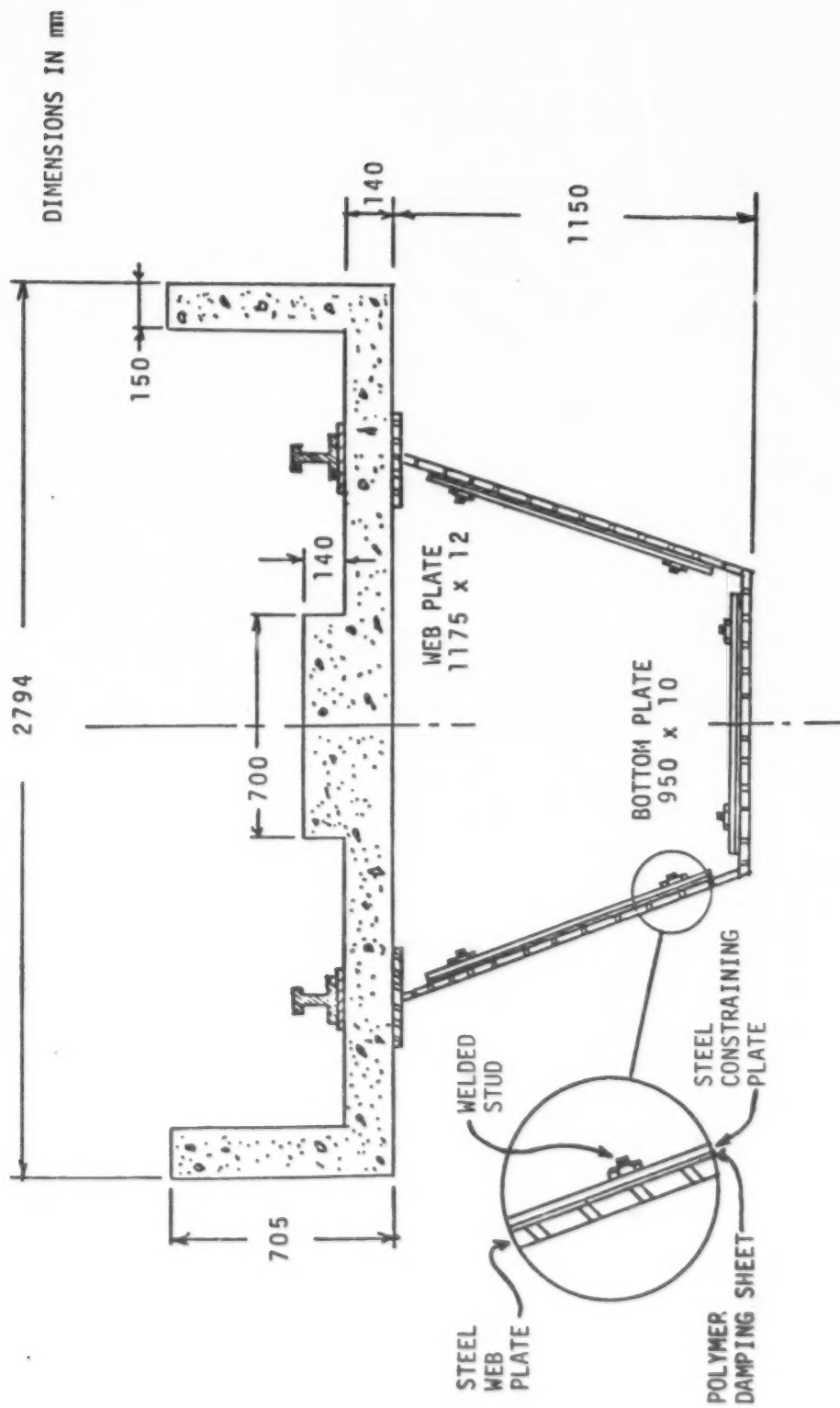


FIGURE 8 CROSS-SECTION OF ICS STEEL/CONCRETE GUIDEWAY BEAM SHOWING THE CONSTRAINED LAYER DAMPING TREATMENT

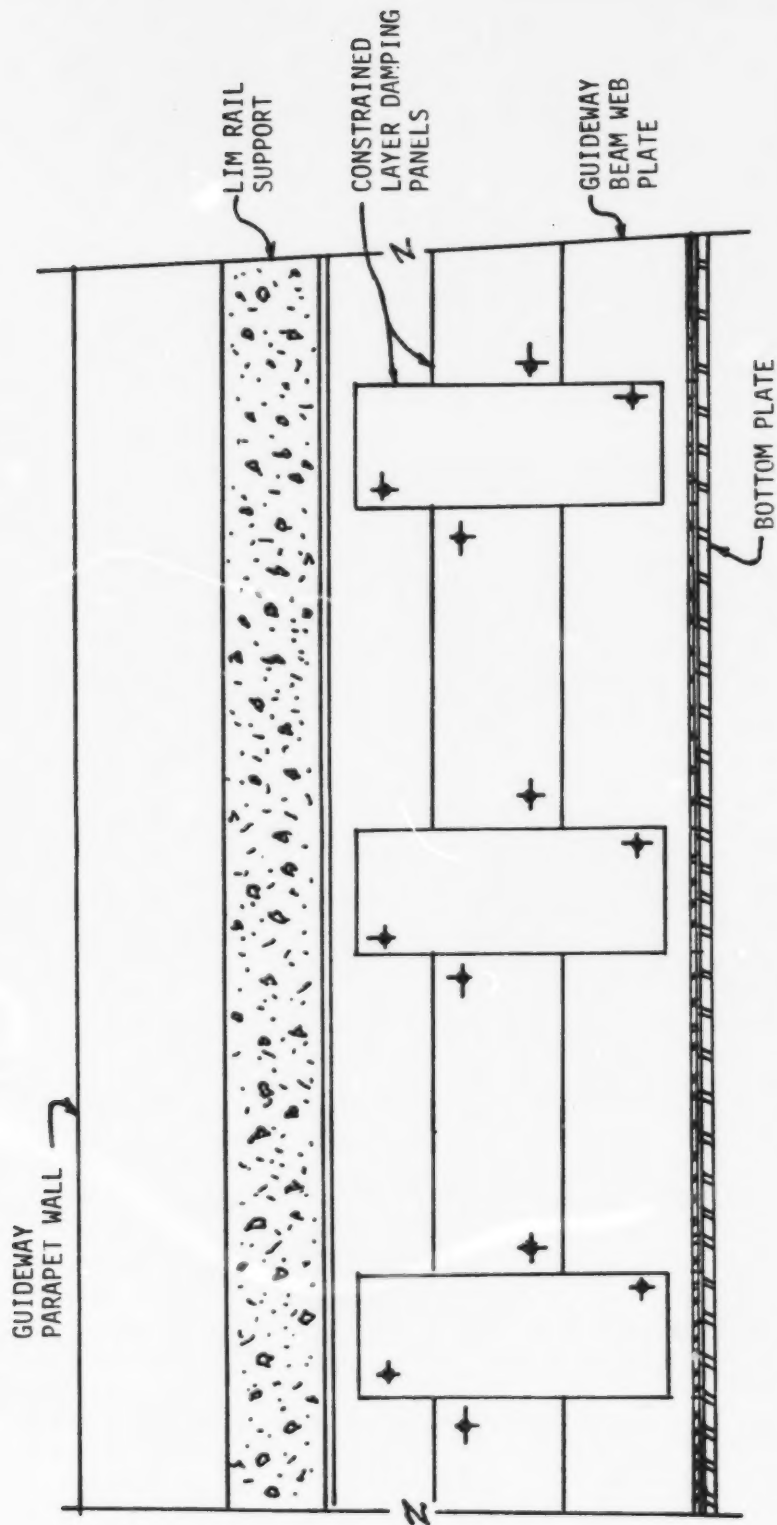


FIGURE 9 ELEVATION SECTION OF ICTS STEEL/CONCRETE GUIDEWAY BEAM SHOWING THE
CONSTRAINED LAYER DAMPING TREATMENT

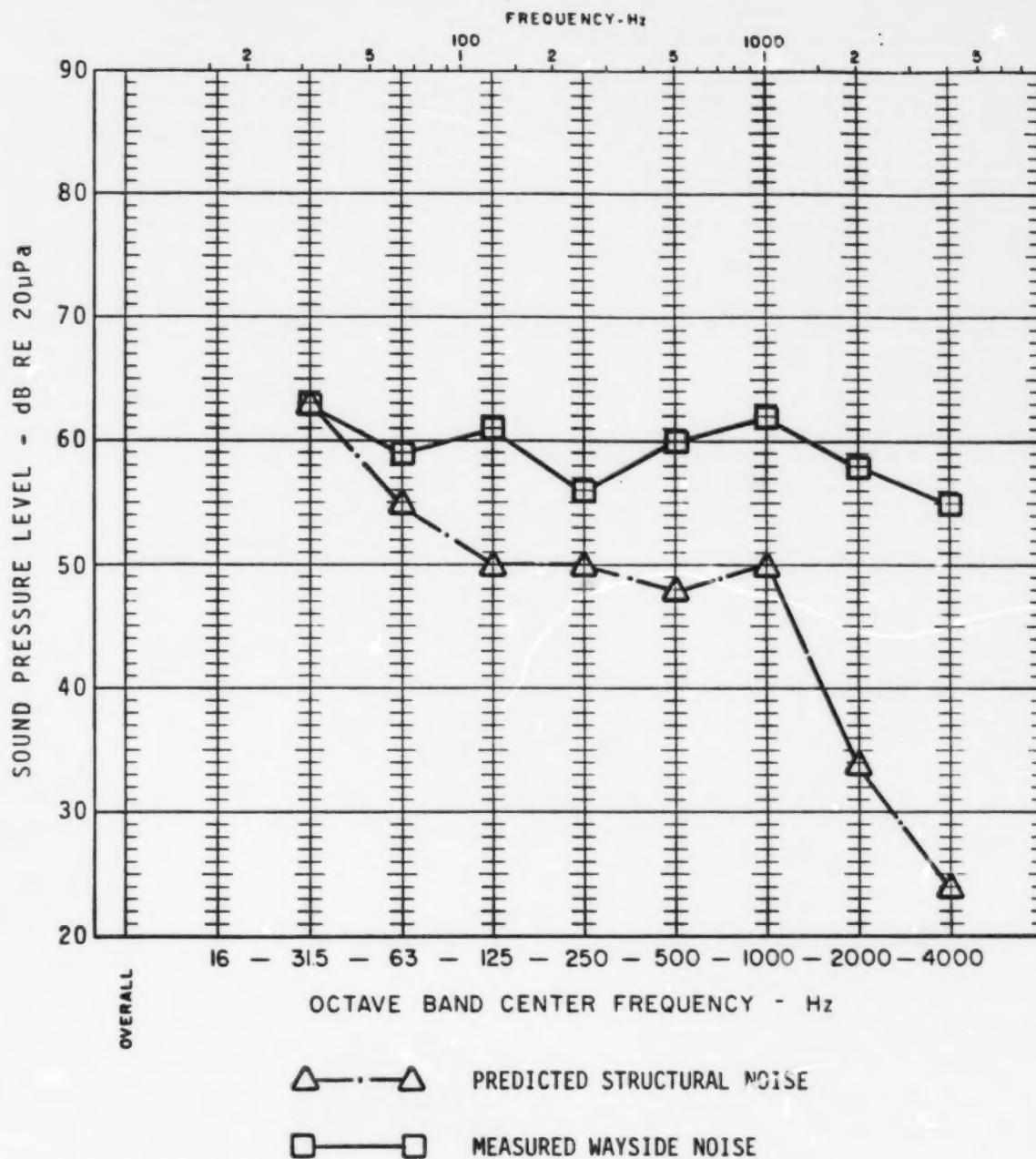


FIGURE 10 WAYSIDE NOISE 15 m FROM ALL-CONCRETE ICTS AERIAL GUIDEWAY AT GUIDEWAY HEIGHT - SINGLE VEHICLE, 72 km/HR ON POLISHED RAIL

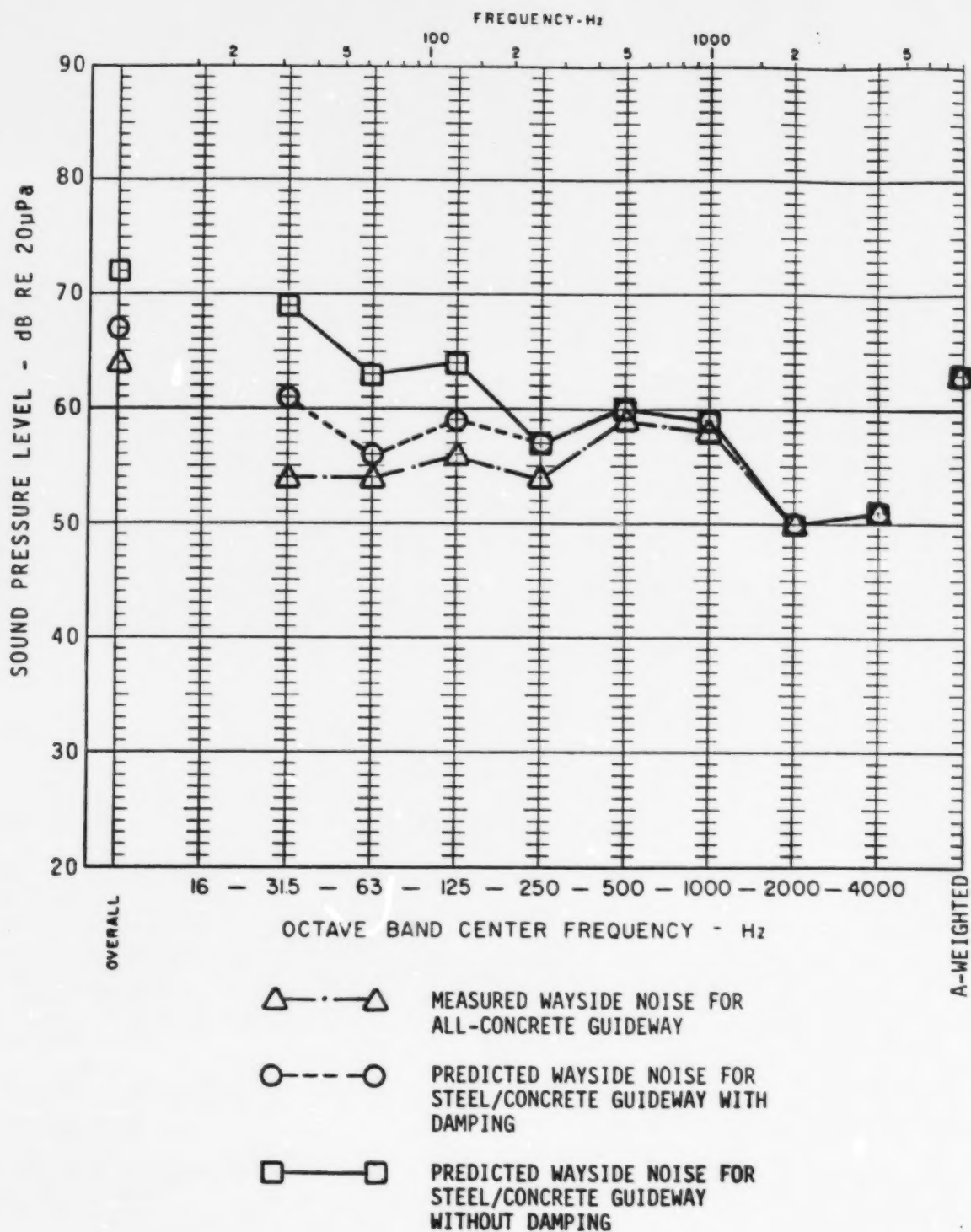


FIGURE 1. COMPARISON OF WAYSIDE NOISE AT GUIDEWAY HEIGHT
15 m FROM ICTS AERIAL GUIDEWAY - SINGLE VEHICLE
54 km/HR ON POLISHED RAIL

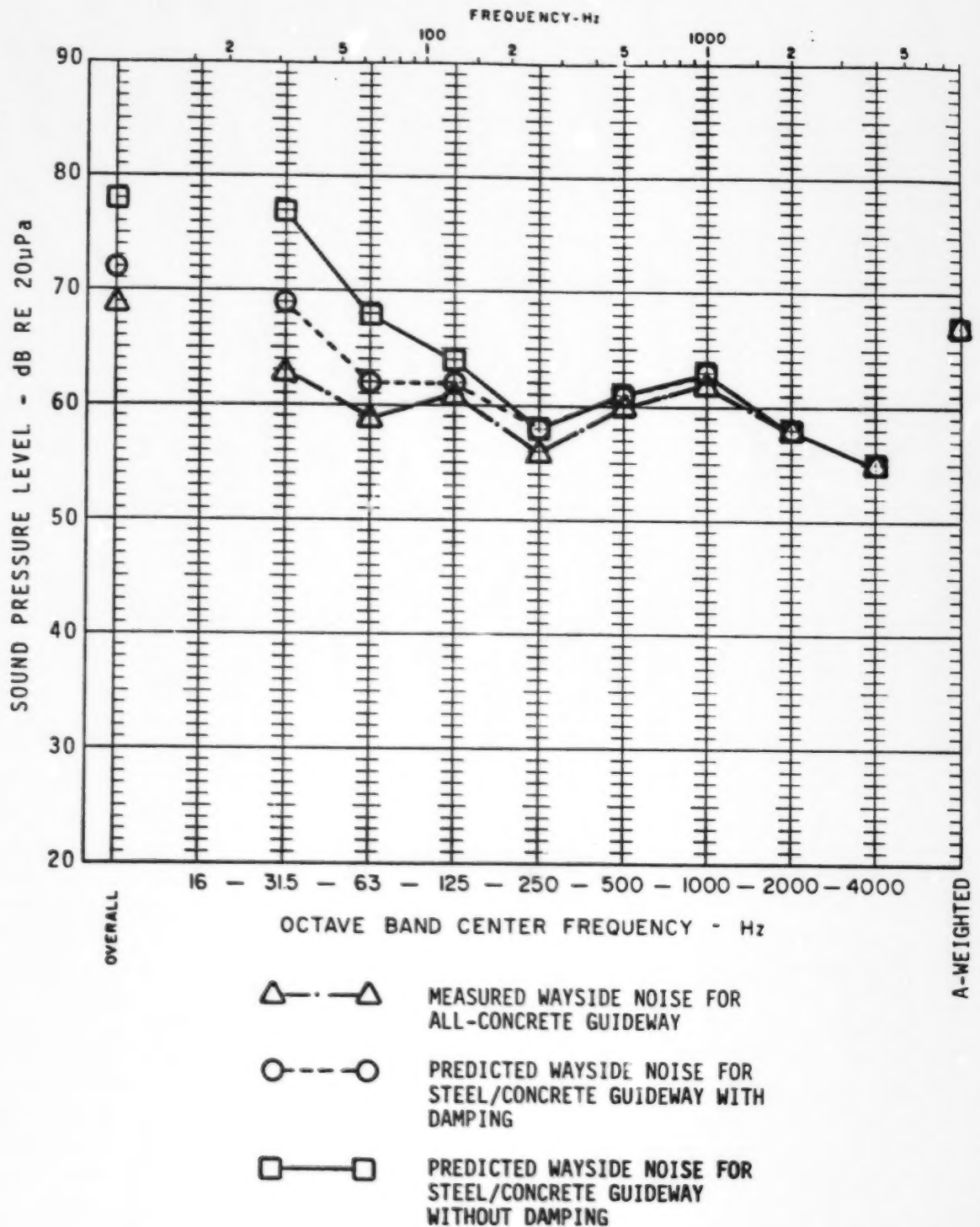


FIGURE 12 COMPARISON OF WAYSIDE NOISE 15 m FROM ICTS AERIAL GUIDEWAY AT GUIDEWAY HEIGHT - SINGLE VEHICLE 72 km/HR ON POLISHED RAIL

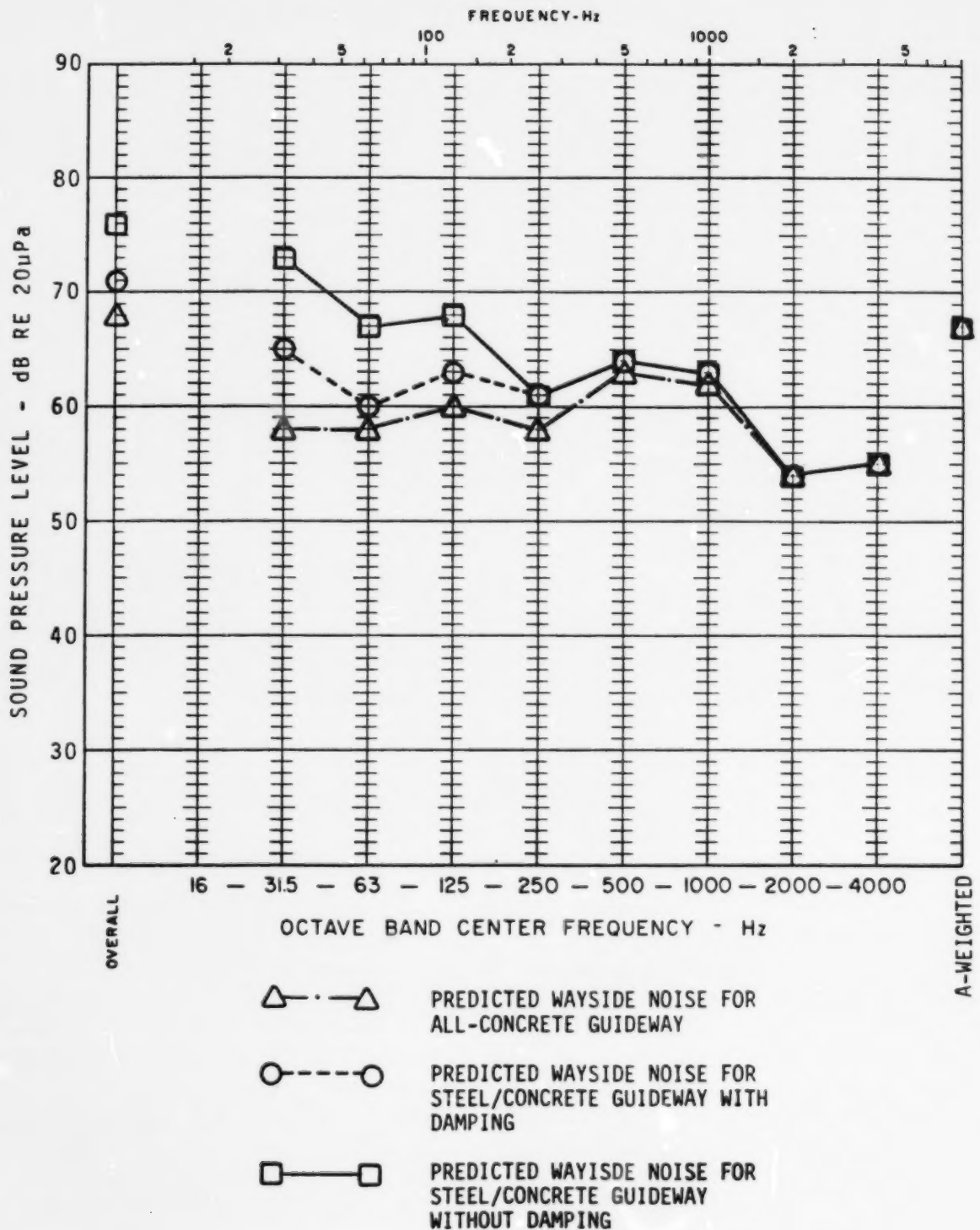


FIGURE 13 COMPARISON OF WAYSIDE NOISE 15 m FROM ICTS AERIAL GUIDEWAY AT GUIDEWAY HEIGHT - 4-CAR TRAIN 54 km/HR ON POLISHED RAIL

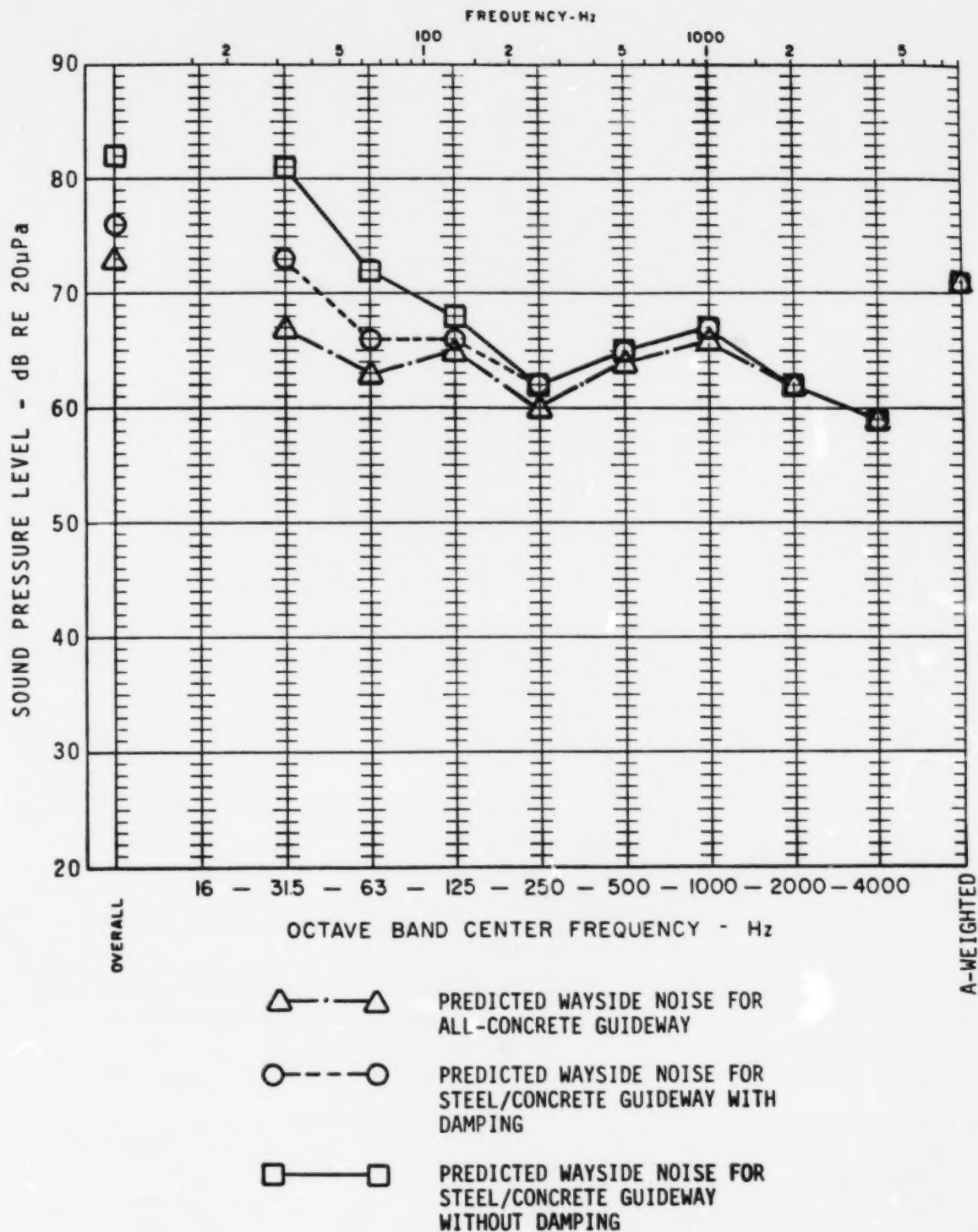
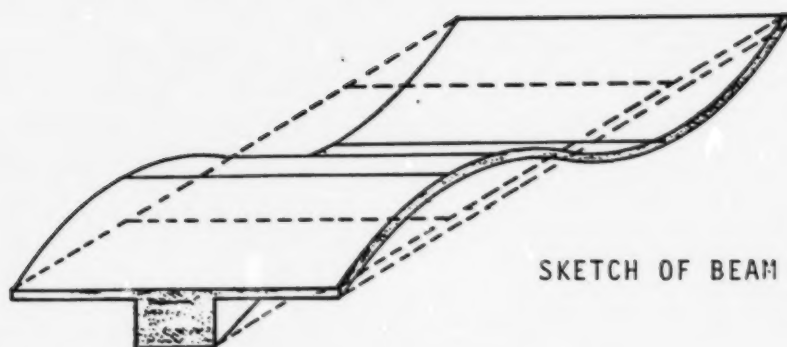
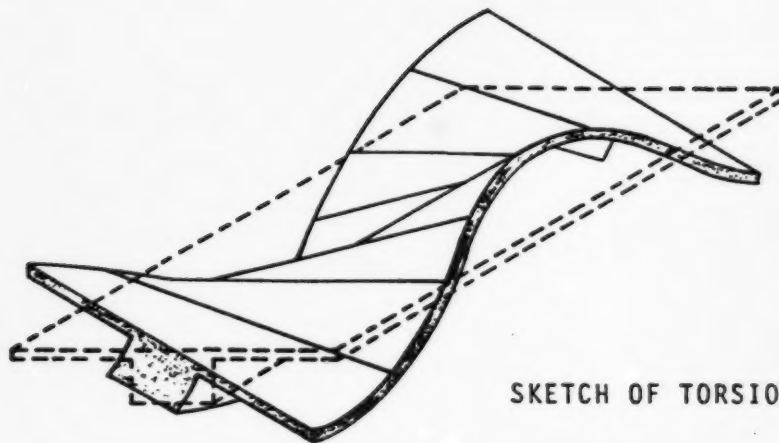


FIGURE 14 COMPARISON OF WAYSIDE NOISE 15 m FROM ICTS AERIAL GUIDEWAY AT GUIDEWAY HEIGHT - 4-CAR TRAIN 72 km/HR ON POLISHED RAIL



SKETCH OF BEAM MODE



SKETCH OF TORSION MODE

REFERENCES

1. S. K. Oleson, "Aerial Structure Noise Measurements", Technical Report, Stanford Research Institute, August 1965.
2. S. K. Oleson, "Aerial Structure Noise Comparisons", Technical Report, Stanford Research Institute, October 1965.
3. V. Salmon and S. K. Oleson, "Noise Control in the Bay Area Rapid Transit System, Final Report", Stanford Research Institute, July 1966.
4. "San Francisco Bay Area Rapid Transit District Demonstration Project Technical Report No. 8, Acoustic Studies", Parsons Brinckerhoff-Tudor-Bechtel, June 1968.
5. G. P. Wilson, "Aerial Structure Noise and Vibration Measurements", Wilson, Ihrig & Associates, Inc., Oakland, California, October 1966.
6. G. P. Wilson, "Diablo Test Track Noise and Vibration Measurements", Wilson, Ihrig & Associates, Inc., Oakland California, June 1967.
7. G. P. Wilson, "Smooth Ground Rail Noise and Vibration Measurements - Diablo Test Track", Wilson, Ihrig & Associates, Inc., Oakland, California, September 1968.
8. G. P. Wilson, "BARTD Prototype Car 107 Noise Tests", Final Report, Wilson, Ihrig & Associates, Inc., Oakland, California, June 1972.
9. Long, Dunlap, Appleton, Gupchup, Beall, and Scheffey, "Dynamics of Vehicle-Structure Interaction, Rapid Transit Structures", Bechtel Corporation.
10. G. P. Wilson, "Aerial Structure Noise for the Washington Metropolitan Area Transit Authority Metro System", June 1972.
11. G. P. Wilson, "Reduction of Noise Radiated by the Walnut Creek Bridge Structure", Wilson, Ihrig & Associates, Inc., Oakland, California, November 1973.
12. E. L. Wilson, "Dynamic Analysis of Seventy Foot Aerial Girder", April 1975.
13. H. J. Saurenmen, "Vibration and Noise Control Recommendations for Aerial Structures for the Metropolitan Atlanta Rapid Transit Authority", Wilson, Ihrig & Associates, Inc., Oakland, California, September 1975.
14. G. Unsworth and L. Gray, "Steel Guideway Study - Noise/Vibration Baseline Data", March 9, 1981.

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.5

ENVIRONMENTAL ASSESSMENT

METRO CANADA LIMITED,
RAPID TRANSIT PROJECT OFFICE,
HAMILTON, ONTARIO

JUNE, 1981

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

ENVIRONMENTAL REVIEW (Preliminary Work Statement (i) 3)

INTRODUCTION

The purpose of this report is to document the differences of impact on the natural environment between the concrete and steel guideway alternatives. The methodology adopted in the comparison was to first, list the environmental parameters already investigated in the "Existing Corridor Environment Report" prepared as Task E-1 of the ICTS Pre-Implementation Program for the Region of Hamilton-Wentworth. Second, to tabulate and comment on the areas where a steel guideway alternative might differ in extent and type of environmental impact. Third, to summarize and comment on the relevant differences between guideway type.

1.0 Existing Corridor Environment

The following bio-physical conditions were documented for the study corridor in the Task E-1 "Existing Corridor Environment Report".

Bio-physical Parameters

- | | |
|------------------|--|
| Air Quality | - micro-macro ambient air quality as monitored by MOE, Stelco, and the City of Hamilton. |
| | - Co, Co ₂ , So ₂ , particulate, etc. |
| Climatic Factors | - Temperature |
| | - wind-force, direction frequency |
| | - sunshine |
| | - precipitation, snow, rain |
| | - micro climate - sun, shading |
| Wildlife | - species and quantities |
| | - food sources, habitat |
| | - movement |

- significance - rare, endangered, threatened species
- barriers
- corridors
- Hydrology
 - ground water
 - drainage
 - surface water - if any
 - water quality
- Topography
 - slopes and slope stability (slides, slumps)
 - topographic features
- Geology
 - composition/formation
 - characteristics
 - bedrock
 - bearing strength
- Soils
 - soil type, structure
 - permeability
 - depth
 - plasticity
 - bearing strength, foundation capacity
 - erosion or erosion tendencies
 - compaction and settling
 - agricultural capacity
 - movement of talus
- Vegetation
 - species
 - condition
 - age
 - height
 - significance
 - barriers
 - corridors

2.0 Steel versus Concrete Guideway Impacts

2.1 Air Quality

2.1.1 Micro Ambient Air Quality - Higher level of Co, Co₂, and dust and particulates will occur during construction of both guideways.

2.1.2 Macro Ambient Air Quality - It is unknown if an increased demand for structural steel would alter macro air quality.

2.2 Climatic Factors

2.2.1 Temperature - Although the thermal properties of concrete and steel differ, the concrete decking over the steel trapazoidal span should offer a heat sink to prevent heat scalding of vegetation adjacent to the guideway.

2.2.2 Wind-force, Direction, Frequency - No differences are anticipated.

2.2.3 Shading - No differences are anticipated since spatial dimensions for each guideway alternate are similar.

2.3 Wildlife

2.3.1 Movement, Barriers, Corridors - Both guideway types are elevated and therefore minimize impact on movement of terrestrial wildlife. No differences are anticipated.

2.4 Hydrology

2.4.1 Groundwater - Although spatial dimensions of the footings for a steel guideway may be different than those for concrete, the impact on groundwater is not significantly different.

2.4.2 Drainage - No differences are anticipated.

2.4.3 Surface Water - No differences are anticipated.

2.4.4 Water Quality - No differences are anticipated.

2.5 Topography

2.5.1 Slopes and Slope Stability - no difference in impacts are anticipated.

2.5.2 Topographic features - all alignments will be constructed in tunnel to minimize impact on the Niagara Escarpment. No differences in impacts are anticipated.

2.6 Geology

No differences in impacts are anticipated.

2.7 Vegetation

No differences in impacts are anticipated.

2.8 Construction Impact

The sequences for foundation construction are similar for both structures. Since the construction of foundations is the most environmentally disruptive aspect in the schedule the remaining differences in the construction time for the erection of a steel box beam or concrete beam tend to balance out.

3.0 SUMMARY OF RELEVANT DIFFERENCES

Bio-physical impacts are similar for both guideway types. No substantive differences in these impacts will occur with the utilization of more structural steel since the basic dimensions and support intervals remain the same.

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.6

RISK ASSESSMENT

METRO CANADA LIMITED,
KINGSTON, ONTARIO

JUNE, 1981

STEEL ALTERNATIVE GUIDEWAY STUDY
RISK ASSESSMENT

TR 122 600 003

THESE TECHNICAL DATA AND THE DESIGNS DISCLOSED HEREIN ARE THE EXCLUSIVE PROPERTY OF METRO CANADA LTD OR CONTAIN PROPRIETARY RIGHTS OF OTHERS AND ARE NOT TO BE USED OR DISCLOSED TO OTHERS WITHOUT THE WRITTEN CONSENT OF METRO CANADA LTD. THE RECIPIENT OF THIS DOCUMENT, BY ITS RETENTION AND USE, AGREES TO HOLD IN CONFIDENCE THE TECHNICAL DATA AND DESIGNS CONTAINED HEREIN. THE FOREGOING SHALL NOT APPLY TO PERSONS HAVING PROPRIETARY RIGHTS TO SUCH TECHNICAL DATA OR SUCH DESIGNS TO THE EXTENT THAT SUCH RIGHTS EXIST.

metro canada limited _____

DATE

PREPARED J. E. Chatfield
J.E. Chatfield, Manager, System Design

CHECKED L.W. Gray
L.W. Gray, Manager, Product Development

APPROVED R. Smith
R. Smith, Director, Engineering

15 June/81

METRO CANADA LIMITED

Page: (i)

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

SUMMARY

A study has been carried out by a team of consultants reporting to Metro Canada Limited in Hamilton to provide inputs to a feasibility report on a steel, or composite steel and concrete, guideway for ICTS.

Metro Canada Limited in Kingston was tasked with identifying any interface problems between the steel guideway and the guideway mounted subsystems and to carry out a risk assessment, based on in-house data and the inputs of the other participants to this study.

As a result of this activity no significant technical problems were identified in the integration of the guideway mounted subsystems with the proposed composite steel beam and concrete deck guideway.

A number of risk areas were identified but after investigation most were reduced to solvable design problems. This left only two areas of risk for which there were no obvious technical solutions. These are the areas of aesthetics and costs.

Aesthetic problems are the staining of adjacent concrete structures with oxides washing off the weathering steel, and a perception that a weathering steel guideway will not harmonize with the concrete, masonry and glass of the CBD. Until these problems are satisfactorily resolved the acceptability of a weathering steel guideway is at risk.

Although the Cole, Sherman and Associates comparative cost estimates indicated an advantage of 3½% for the steel composite guideway it is felt that a more in-depth study is required before cost data is available which could form the basis for evaluating the merits of composite versus all-concrete guideway. Until firmer cost data is available the achievability of the predicted guideway cost figures is at significant risk.

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

TABLE OF CONTENTS

SUMMARY

1.0	INTRODUCTION
2.0	SCOPE OF WORK
3.0	SUBSYSTEM INTERFACES
4.0	RISK ASSESSMENT
5.0	CONCLUSIONS
6.0	RECOMMENDATIONS

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

1.0

INTRODUCTION

In connection with a proposal to provide an Intermediate Capacity Transit System for the Hamilton area, the region of Hamilton-Wentworth initiated a study to assess the viability of using steel, or a composite steel and concrete, guideway instead of the reinforced concrete guideway initially developed for ICTS application.

Metro Canada Limited in Hamilton were tasked with the organization of this study. A number of consultants were given specific tasks in support of this effort, the lead role going to Cole, Sherman and Associates Limited to provide inputs on cost, feasibility, constructability and aesthetics. Wilson, Ihrig and Associates provided an analysis of radiated noise and damping requirements. A literature survey in support of this study was carried out by MCL Hamilton. ABAM Engineers, Inc., designers of the prestressed concrete ICTS guideway, provided the guideway design criteria for this study.

Metro Canada Limited in Kingston were tasked with supporting the study by considering the subsystem interfaces with the guideway and by making an overall assessment of the risks involved in adopting steel as the primary guideway construction material. The results of this evaluation are the subject of this report.

METRO CANADA LIMITED

Page: 2

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

2.0 SCOPE OF WORK

2.1 Subsystem Interfaces

Use of MCL Kingston resources to study subsystem interfaces with the proposed steel guideway, with special attention to any perceived interference with the ATC data communication via the in-track inductive loop and antenna system.

2.2 Risk Assessment

Use of MCL Kingston resources and data bank to assess perceived risks that may arise as a result of the steel guideway proposed with special attention to the sound radiation characteristics of the steel guideway. The task included the provision of acoustic data to the consultants, Wilson Ihrig and Associates, who were tasked with assessing the Steel Alternative Guideway Acoustic Analysis.

2.3 To review the reports prepared by the consultants who were contributing to the steel guideway study.

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

3.0 SUBSYSTEM INTERFACES

3.1 General

There are a number of ICTS subsystems interfacing with the guideway including the following:

- a) trackwork, including special trackwork switches and switch machines;
- b) power rails, including insulators and attachments;
- c) LIM reaction rail and attachments;
- d) walkway for servicing and passenger evacuation;
- e) lighting, emergency phones, power rail emergency switches;
- f) conduits for auxiliary power and signal cables;
- g) inductive loop for ATC data communications.

3.2 Evaluation of the physical location and means of attachment of each of the above subsystems to the guideway indicates that location and probable means of attachment is such that the proposed composite steel beam, using a reinforced concrete deck and sidewalls, presents no fundamental problem in the attaching and locating of the identified subsystems within the guideway.

This conclusion follows from the observation that the part of the guideway where all these subsystems are currently planned is of essentially similar shape and of similar materials as that would be used for the ABAM developed all concrete guideway. It follows, therefore, that no fundamental problems should arise in the interfacing of these subsystems with the guideway.

METRO CANADA LIMITED

Page: 4

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

However, in the attachment of some of these subsystems to the guideway there are indications of a different approach proposed for the steel composite guideway to that proposed for the all-concrete guideway. This appears to be due to a perceived risk involved in achieving the required tolerances for the guideway mounted subsystems by the process of casting attachments and seatings for these subsystems during the precasting stage. If it is not possible to cast these attachments in place during the pre-casting of the concrete deck of the steel guideway, total costs of the installed system may be significantly increased. It can be argued that the perceived risk in precasting the attachments in the deck is equally applicable to the ABAM concrete guideway. However, it is noted that the ABAM guideway has been the subject of fixed construction estimates which gives some confidence that, in the case of the concrete guideway, this process is not perceived as a risk area.

Further investigation of the methods of installing guideway-mounted subsystem attachments is required.

A particular aspect of subsystem integration that was to be investigated during this study was the potential impact of the steel guideway on the data communication system used for the Automatic Train Control (ATC) system.

The data communication system between the wayside control and the Vehicle Onboard Control system consists of an inductive loop mounted between the running rail with the transmitting and receiving antennas mounted on the vehicles. The concern was that electro-magnetic fields generated in the steel guideway would corrupt the data being transmitted to and from the vehicles, thus resulting in reduced reliability of operation.

METRO CANADA LIMITED

Page: 5

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

Investigation of the possibility of data corruption of the messages being transmitted via the inductive loop has confirmed that considerable testing of this data communication has been conducted in Germany, including operation on long span steel bridges. These tests and demonstrations have provided a high degree of confidence that the use of a steel guideway for ICTS would not result in an increase in corruption of data being transmitted to and from the vehicles and wayside by the automatic train control system. This has also been discussed with SEL Canada, suppliers of the ATC equipment for the ICTS. As a result it is concluded that for the composite guideway, as long as the re-bar density did not exceed that used at the Kingston test track elevated guideway, and that no large pieces of metal crossed between the two running rails at the level of the concrete surface or above it, then the ATC system would be relatively insensitive to the choice of a composite or all-concrete guideway.

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

4.0 RISK ASSESSMENT

4.1 General

The risk assessment carried out by MCL in support of the Steel Alternative Guideway Study is based on the existing MCL data bank and in-house expertise as well as the data provided by the the other participants in this study. It also reflects study of the final reports submitted by the other contributors to this phase of the study.

4.2 Structural Design Risk

It is very obvious from the work that has been done to date that there is no question that the proposed composite steel guideway can be designed to meet all the design loads required by the set of design criteria prepared for the ICTS guideway.

4.3 Guideway Construction

Risks in guideway construction do not seem to be significant except that there is a perception that the steel beam would be erected on the crossheads first, followed by the sections of the concrete deck, and then the elements would be grouted together to form the composite whole. Thus, in effect, two beam sections, one a steel trough and the other the concrete deck and sidewalls would need to be installed for each equivalent single beam of concrete construction. This is not truly a risk area but may impact the time period and extent to which there is disruption of normal access and business activities in the vicinity of the construction site.

The method of joining the steel element to the concrete deck in the case of the composite beam involves a second pour and there would seem to be no viable alternative to this procedure. A great deal of design effort has been addressed to means of avoiding a second pour for track alignment purposes in the case of the precast concrete beam. However, given that a second pour is required to unite the concrete and steel elements of the composite beam it

METRO CANADA LIMITED

Page: 7

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

is appropriate and good practice to utilize the same second pour for alignment of the running rail fasteners as proposed in the Cole, Sherman and Associates report.

However, this does not address the means of alignment of other guideway mounted components such as the LIM reaction rail and power rails. If the fasteners for these elements have also to be set by a second pour in order to achieve the desired tolerances, significant additional costs may be involved. In addition to the increased installation cost by this method, redesigned and thereby significantly more costly support insulators may be required for the power rail installation.

4.4

Aesthetics

There appears to be at least two problems with the steel composite guideway that relate to aesthetics. The first of these would appear to be the staining of adjacent structures by the products of oxidation during the early months of exposure of the proposed weathering steel. The second is the perception that the public may find the appearance of weathering steel less than harmonious with the typical downtown surroundings of concrete, masonry and glass. At the same time, it is suggested that weathering steel may have visual advantages in natural and residential surroundings.

In regard to the potential staining of adjacent structures by oxidation drippings from the weathering steel prior to the formation of the final weather proof oxide film on the steel surface, it has been stated in the Cole, Sherman report that "run-off of iron oxides will occur to a major degree during the first two or three years and will continue at a greatly reduced rate for the life of the installation". Also that "the successful, 100% elimination of staining due to run-off cannot be guaranteed".

METRO CANADA LIMITED

Page: 8

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

It has also been pointed out that weathering steel requires great care in handling during the construction period in order that uniform weathering occurs, and that sandblasting after the completion of construction may be necessary.

There does not appear to be any certain method of protection against this problem of staining. A Japanese process being investigated by Stelco requires cleaning of the steel surface, then application of a proprietary solution, followed by spraying with an acrylic resin. The effectiveness and cost of this solution has yet to be established.

On the basis of the data currently available it would appear that the problem of staining adjacent concrete and masonry structures with the run-off from the weathering steel guideway could seriously detract from the attractiveness of the guideway system and its surroundings and thus community acceptance of ICTS. Pending further investigation of the effectiveness and cost of potential solutions, this must be considered as a high risk area.

On the subject of weathering steel being less than harmonious with the downtown surroundings it has been stated that "following sufficient exposure to appropriate conditions, weathering steel takes on a rich, dark brown, rough textured appearance". In industrial atmospheres this could become almost blue in time. With the Japanese pre-weathering process it is said to be possible to generate a final colour in the dark grey-blue range.

It is suggested in the Cole, Sherman report that these colour combinations may not be harmonious with the downtown background and that the weathering steel guideway may be better applied in natural and residential surroundings. The implication that residential areas would find the appearance of weathering steel attractive could be questioned.

METRO CANADA LIMITED

Page: 9

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

Of more significance is that it has been established that business and commerce expands into low density areas served by good transportation. It is anticipated that any low density areas existing prior to construction of ICTS would become high density residential with centres of business and commerce along the total route during the expected life of the ICTS system.. Thus, the continued existence of natural or low density residential areas could be a fallacy.

On the basis of the opinions on aesthetics expressed in the Cole, Sherman report it is concurred that the composite guideway of weathering steel may be less acceptable than concrete in developed urban areas. Furthermore, it is contended that successful application of ICTS will create development along substantially all of the ICTS route and thus there is a risk of adverse aesthetic impact from the use of steel guideway anywhere along the ICTS route subject to such development. Until these aesthetic problems are satisfactorily resolved the acceptability of a weathering steel guideway is at risk.

The Cole Sherman report asserts that the introduction of curved radiused forms rather than the hard edges and cornered rectangles currently indicated would reduce the visual impact of the guideway. Apart from the subjective nature of this observation (other studies have drawn the opposite conclusion) it is noted that such curved, radiused forms would be more easily achieved in concrete than in the welded 12 mm steel plate of the composite beam.

4.5

Acoustics

The Wilson, Ihrig and Associates report on the acoustic analysis of the proposed composite steel and concrete beam indicates that the main concern with the composite beams vis-a-vis the concrete beam is in the area of low frequency vibration. However, they indicate that constrained layer damping can be applied to the steel plates of these beams, thereby providing essentially the same acoustic performance as the all-concrete beams. The estimated cost of this damping would be \$170/metre and represents a significant cost penalty.

METRO CANADA LIMITED

Page: 10

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

Wilson, Ihrig indicate that in the CBD undamped composite beams would be acceptable. This is because of the heavy masonry construction of buildings in the CBD and the existence of other sources of low frequency noise. (It will be recalled that the Cole, Sherman report indicated that the composite guideway may not be acceptable in the CBD on aesthetic grounds.) In other areas where there are no lightweight residential dwellings, undamped composite guideway also would be acceptable. In areas where there are a significant number of residences with associated night-time occupancy composite beams would need to be of the damped configuration to avoid the low frequency rumble that would result in dissatisfaction and complaints.

From the work of the acoustic impact consultants it is evident that the composite beam can be brought to a satisfactory performance level but at a significant cost penalty.

4.6

Cost

The cost comparison activity carried out by Cole, Sherman and Associates has indicated that the cost of a typical guideway alignment could be 3½% less for the composite beam with the precast deck than for the all-concrete guideway. That the figures for both systems are so close means that such figures cannot be used in the decision making process at this time. Further investigation will be required before the cost data can be made available on which a firm decision can be taken.

Areas in which significant cost increments may increase the cost of the composite guideway structure are:

- a) Additional costs that may be incurred in the manufacture of guideway mounted hardware to accommodate different mounting provisions inherent in the construction process of the composite guideway;

METRO CANADA LIMITED

Page: 11

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

- b) Additional costs that may be incurred in the installation of guideway mounted hardware required to accommodate different mounting provisions inherent in the construction process of the composite guideway.
- c) Additional costs incurred as a result of special processes required to ensure the rapid formation of the desired oxide coating on the weathering steel.
- d) Costs incurred in any special processes required to protect concrete and masonry adjacent to the steel guideway from discolouration from the weathering steel.
- e) Costs incurred in applying acoustic damping to the panels of composite beams to reduce low frequency rumble.

It must also be accepted that the comparative cost study carried out by Cole, Sherman could hardly be exhaustive in the given time frame. In particular, although the pre-cast concrete guideway cost figures were obtained in the Hamilton region, as were the cost figures for the composite guideway (and thus should make for a sound comparison) it should be noted that the all-concrete beam costs obtained are significantly higher than data provided to MCL by ABAM. While recognizing that the ABAM data may not be strictly comparable it will be noted that ABAM have carried out a number of cost estimates in connection with firm bids on the all-concrete ICTS beam. This should give some credence to the cost data provided by ABAM for the pre-cast type of beam. Further work is necessary to ensure that both sets of cost figures properly reflect the costs of the installed ICTS guideway. Until such firmer cost data is available the achievability of the predicted guideway cost figures is at significant risk.

METRO CANADA LIMITED

Page: 12

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

5.0 CONCLUSIONS

It is concluded that there are no areas in which integration of guideway mounted subsystems and interfaces cannot be successfully achieved with the proposed composite steel and concrete guideway although there may be cost penalties relative to integration with the all-concrete guideway..

No problems are anticipated with the ATC data communication system arising from the use of the proposed composite steel and concrete guideway.

There are no perceived guideway design, manufacture or construction risks associated with the composite steel and concrete guideway.

There is a significant risk of staining adjacent concrete, masonry and brickwork with the oxides of iron washed from the weathering steel guideway pending the achievement of a stable oxide film. A Japanese process to accelerate the formation of a stable oxide film is being evaluated by Stelco but no data is available on its effectiveness or cost of application at this time.

There is a perception that the composite steel guideway will not harmonize with the concrete, masonry, glass and asphalt of the CBD.

On the basis of the data provided no problems are anticipated in achieving satisfactory damping of low frequency vibration using constrained layer damping on the steel panels of the composite guideway. In guideway areas where damping is required there will be a cost penalty.

There is a perception that the comparative cost figures derived from the all-concrete and composite guideway, which show a 3½ cost advantage for the composite guideway system in the selected scenario, may be modified when account is taken of:

- a) manufacture and attachment of guideway mounted subsystems:

METRO CANADA LIMITED

Page: 13

Report: TR 122 600 003

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

- b) protection against staining adjacent concrete structures with oxides of iron;
- c) provision of acoustic damping on the composite guideway.

A number of proposals are presented in the Cole, Sherman report which may reduce the cost of the composite guideway.

Subject: STEEL ALTERNATIVE GUIDEWAY STUDY - RISK ASSESSEMENT

6.0 RECOMMENDATIONS

Further in depth analysis is recommended in future phases of this study in the areas of:

- a) resolution of the staining problem;
- b) detail design of guideway mounted subsystems to suit the construction methods proposed for the composite beam;
- c) more detailed design of representative steel beams, including in depth structural analysis;
- d) more detailed cost estimates of the fabrication and installation of the composite guideway;
- e) more detailed cost estimates of the fabrication and installation of the all-concrete guideway in the Hamilton area.

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

APPENDIX 8.7

WORK STATEMENTS

METRO CANADA LIMITED,
RAPID TRANSIT PROJECT OFFICE,
HAMILTON, ONTARIO

JAN. 1981

HAMILTON-WENTWORTH RAPID TRANSIT PROJECT

STEEL ALTERNATIVE GUIDEWAY STUDY

WORK STATEMENTS

(i) Management and Environment - MCL Hamilton

1. Review of previous studies and preparation of a résumé, analysis and summary report.
2. Co-ordinate and review all aspects of the feasibility study team activities.
3. Review all environmental aspects of the steel guideway alternative except noise and vibration. Determine the impact of the Environmental Assessment Act on the study.
4. Evaluation of the study results and preparation of the feasibility report.

(ii) Systems Review - MCL Kingston

1. Risk assessment and evaluation.
2. Sub-system interface considerations.
3. Monitor design criteria input to structural team.
4. Investigate and propose interference reduction techniques for SELTRAC ATC sub-system.
5. Prepare report on systems review as an appendix to the feasibility report.

(iii) Information and Criteria - ABAM

1. Supply latest design criteria as required by all teams.
2. Collaboration with other teams during entire study.
3. Prepare report on information and criteria as an appendix to the feasibility report.

(iv) Structural and Aesthetics - COLE SHERMAN

1. Investigate the trapezoid proportions discussed in the MTC paper dated April, 1979. Determine the appropriate section for a trapezoidal shaped steel guideway straight beam applicable to a span of 100 feet + 20 feet and to the maximum number of special beam configurations.
- 2. Develop preliminary design concepts for a variety of span configurations, using standard trapezoidal steel section (straight, constant-radius curves, horizontal and vertical spiral curves, crossover and other special configurations). Assess economics and prioritize.
3. Develop detailed designs in priority sequence.
4. Review aesthetics, particularly weathering (with C.I.S.C. assistance) and noise and vibration control system with noise and vibration consultant.
5. Compare selected trapezoidal steel section with trapezoidal concrete section for cost, supply and delivery and constructability.
6. Cost comparison of steel vs. concrete span configurations which are considered as possibilities in typical alignment segments.
7. Prepare report on structural and aesthetics as an appendix to the feasibility report.

(v) Noise and Vibration - WILSON IHRIG

1. Relate noise and vibration characteristics of proposed steel guideway to concrete guideway to determine radiation characteristics.
- 2. Develop control system for steel guideway noise radiation.
- 3. Relate steel guideway noise and vibration characteristics to three site locations and compare with concrete guideway characteristics (Central Business District: Escarpment: Mountain.)
4. Prepare report on noise and vibration as an appendix to the feasibility report.